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Journal of the
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EDUCATION FOR CONTINUOUS IRRIGATION IN THE HUMID AREAS[a]

Billy B. Bryan[1]

ABSTRACT

Irrigation engineers inevitably serve as irrigation educators. As educators they should be familiar with the factors effecting costs, production and consequent net profits. These include irrigation frequency, land surface preparation, drainage and water distribution. This paper presents an analysis of data concerning these and other factors.

SUMMARY

The degree to which farmers must be educated to make more effective use of irrigation water in the humid area will vary considerably with the location, length of time irrigation has been used, and the amount of educational work which has been done by public and private agencies. Engineers who design irrigation systems and farmers who use these systems must know that in order to obtain optimum yields and maximum net income from using irrigation water, plant population, fertilizer level, varieties, disease control and insect control are all important factors which must be considered and adapted to their particular situation. They must also understand that while these factors are important, the rainfall pattern in the humid areas has been such that the desirable combinations of such items would for most crops be the same for both the irrigated and non-irrigated systems of farming. It should be emphasized that most effective use of the irrigation water itself is of primary importance in obtaining and stabilizing yields and net income at high levels.

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a. Presented to the Fifth National Conference, Irrig. and Drainage Div., ASCE, Memphis, Tenn., September 25, 1958. Published with the approval of the director of the Arkansas Agri Experiment Station.

1. Associate Prof. and Associate Agri. Engr., Univ. of Arkansas, Fayetteville, Arkansas.

The most effective use of irrigation water can be made through the medium of using better irrigation practices and techniques. Among those practices and techniques are the following:

1. Apply water at each irrigation before the plants deplete the moisture in the soil to the wilting point on any part of the area to be irrigated.
2. At each irrigation apply enough water to replenish completely the soil moisture supply to field capacity.
3. Irrigate often enough to maintain an adequate soil moisture supply.
4. Insure that the water is distributed uniformly over the area to be irrigated.
5. Provide an adequate drainage system.

At the invitation of the committee on Irrigation and Drainage in the Humid Area, the author discusses in the paper the extent to which Engineers concerned with the design and utilization of irrigation systems may aid farmers in obtaining most efficient use from irrigation.

Since the magnitude of the farmer's requirement for irrigation in the humid areas is determined directly by the rainfall which occurs during the growing season, it is deemed desirable to develop briefly for the engineers certain background material noting the effect that the normal rainfall pattern has had on recommendations for normal cultural practices. This discussion will also aid in the engineer's understanding of the concept of irrigation which is prevalent among persons in the humid areas of the United States. While data from the Arkansas Agricultural Experiment Station will be cited in support of certain conclusions, it is believed that this data is representative of that accrued at other locations throughout the humid area.

The average quantity of rainfall occurring during the growing season is of itself essentially adequate to sustain optimum yields of most of the crops normally grown in the humid areas. In the East Central Division of Arkansas, the long term average rainfall which occurs during the growing season—April to September, inclusive— is 21.82 inches, or 44 per cent of the long term average annual rainfall.(1) While the quantity of rain which actually occurs and the water requirement of the various crops vary from year to year, this quantity of water is generally considered to be sufficient to sustain optimum yields on the soil, and under the conditions encountered in that section of the state. It cannot be assumed, however, that all of the water which occurs from rainfall on the land enters the soil to become available for plant use. On the contrary, a large portion of the rainfall occurs as intensive, short duration rain which compacts and seals the soil, causing some of the water to run off.

The rainfall which does occur is distributed in such a manner that optimum yields can be expected only infrequently. For example, in only one (1955) of the eight years of experimentation with cotton at Marianna has the distribution of rainfall been such that irrigation did not cause an increase in yields.(2) During that year, three irrigations were made, each of which was followed by rain of significant amounts within five days. In these same cooperative studies with cotton at the Cotton Branch Experiment Station, Marianna, Arkansas, the Agricultural Engineer and Agronomist found it necessary to supply by

irrigation an average of only 24 per cent of the total water made available to the plants. This ranged from a low of 8.7 per cent in 1955 to a high of 45 per cent in 1954.(2) These data indicate that irrigation is of value in supplementing the quantity of rainfall but more important in terms of supplementing the distribution of the rainfall.

The rainfall pattern in the humid areas has enabled researchers to recommend for ordinary farming conditions cropping practices which result in optimum or near optimum production of the major crops within the capability of the various soil classes. Experiments conducted by the Arkansas Agricultural Experiment Station (Departments of Agricultural Engineering and Agronomy of the University of Arkansas cooperating) indicate that the rate of fertilizer application recommended for cotton should be identical for both irrigated and non-irrigated farming conditions.(2) With a few exceptions, the same results have been derived from experiments with other crops. In addition, the recommendations for plant population and variety as well as that pertaining to the relationship between plant population, variety and fertilizer level are the same for the non-irrigated as for the irrigated systems of farming.(2) The exceptions to these general statements are found in the field of Horticultural crops, mainly tomatoes, strawberries and sweet potatoes, where varieties and spacing particularly seem to be factors, and in the case of soybeans where the longer season varieties produce higher yields.

The fact that the normal cropping practices recommendations make no allowances for additional moisture which can be provided by irrigation derive from several reasons:

1. Many factors, not yet isolated and evaluated, other than moisture, fertilizer, plant population, and variety enter into the complex problem of producing optimum yields which are uniform from year to year.

2. Experimental techniques, up to this date, are not adequate to isolate and evaluate the inter-related complex biological and physiological problems which result directly from the weather conditions existing each year.

3. It is believed by experts working in the field of soils that the experiments in Arkansas are conducted on soils whose physical properties limit the maximum production to present maximum yields.

The above discussion should not be construed to indicate that all farmers in the humid area who irrigate are following production practices which will enable them to obtain optimum yields. It is common for farmers to use less than the amount of fertilizer, for example, which is recommended as a result of tests of the soil on their farms. In addition it is common for those farmers irrigating some of the crops where variety is a consideration, to continue to produce that variety which they grew before starting irrigation.

The above discussion points out the need for engineers concerned with the design of irrigation facilities to aid farmers in the humid areas in an understanding that more effective use of the irrigation water itself is of more immediate importance in obtaining optimum yields from crops in the humid area than is a radical change or intensification of other production practices. It is emphasized that irrigation is but another tool for management to use in order to eliminate lack of moisture as a limiting factor in crop production; it is not a substitute for management nor is it a panacea to answer all production problems.

Education for Irrigation

The engineer's role in education for more effective use of irrigation will be discussed under two general headings: (1) An understanding of what irrigation in the humid area can be expected to accomplish in terms of production and (2) improvement of irrigation practices and technique which will result in optimum production.

Role of Irrigation

Effective irrigation can be expected to cause an increase in the yield of many of the crops ordinarily grown in the humid areas. Experiments in Arkansas with cotton,(2) corn,(3) sorghum(4) and strawberries,(5) Fig. 1-4 incl.; and with sweet potatoes, snap beans, and tomatoes, indicate significant increases in yields in every year of the tests. In the case of cotton, a major crop in this part of the humid area, Figure 1, the average increase in yield for the 8-year period 1950-1957, inclusive, is 253 pounds per acre of lint (.51 bales per acre) or 40.7 per cent, with the range from -66 pounds per acre in 1955 to 625 pounds per acre in 1956. In only one year, 1955, did irrigation cause a decrease in the yield of cotton (not statistically significant at the 5 per cent confidence level). It must be recalled that during the growing season of 1955 three irrigations were made on cotton each of which was followed within 2-6 days by rain of significant magnitude which, in effect, nullified any benefit from irrigation.

Irrigation can be expected to result in a tendency for yields to be stabilized at high levels. In the case of cotton, for example, the data, Figure 1, may be used to show that irrigation has reduced the annual deviation from the eight-year mean yield from 214 pounds per acre to 160 pounds per acre. For other crops, such as forage sorghum, the variation in annual yields is even less.

Irrigation can be expected to result in an increase in net income from many of the crops produced in the humid area. The estimated net increase in income resulting from irrigating cotton is cited as an example. The net income and net increase in income which may be reasonably expected from irrigating cotton was estimated by Agricultural Engineers and Agronomists at the Arkansas Agricultural Experiment Station. These estimates were based on yields actually obtained in irrigation experiments, the market prices prevailing each year of the tests, and an investment in irrigation equipment which was considered to represent the maximum which a farmer in Arkansas would encounter. These estimates, Figure 5, show that in 1955, the year most unfavorable to irrigation, irrigation reduced the net income by $40.99 per acre. In 1956, the year most favorable, irrigating cotton resulted in an estimated net increase in income of $133.43 per acre. Since these estimates are derived from all costs connected with the irrigation of cotton (including the annual capital charges associated with an investment in sprinkler irrigation equipment of $155.87 per acre) the increase in net return during this year alone would almost pay for the equipment. The average increase in net returns attributable to irrigation for the eight-year period, 1950-57 inclusive, is estimated to be $44.68 per acre.

Improved Irrigation Technique

The most effective use of irrigation water in the humid areas can be attained through the employment of better and improved water application practices and techniques.

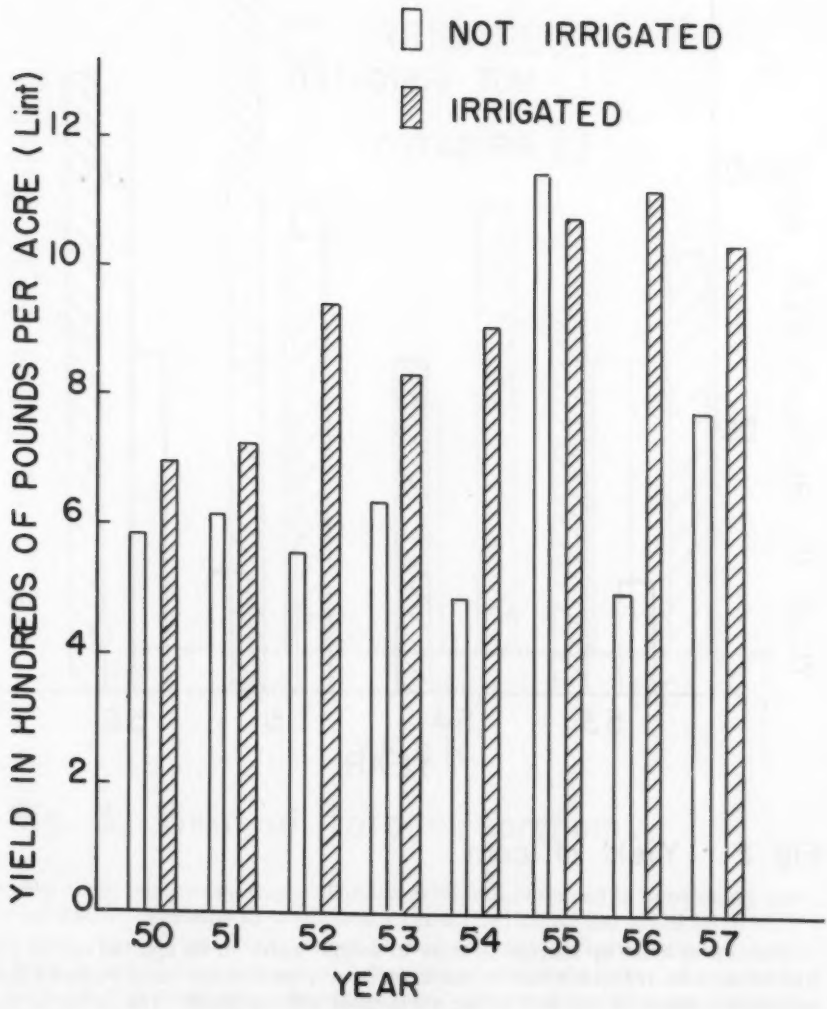

Fig. 1. Yield of cotton.

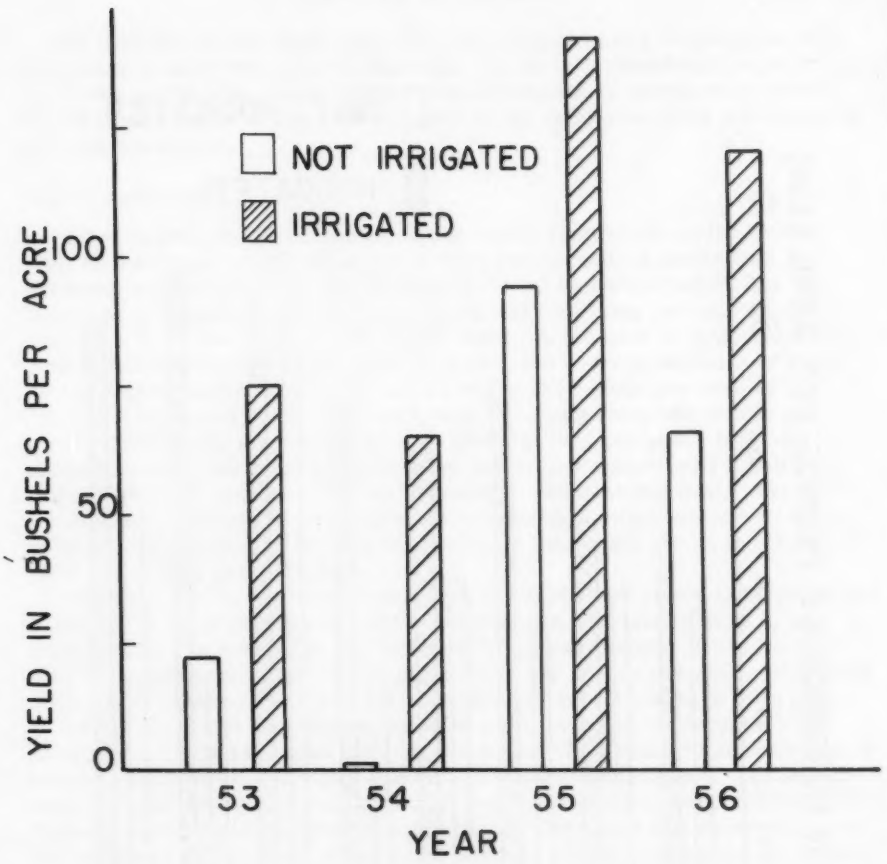

Fig. 2. Yield of corn.

Irrigation must be started in time to enable water to be applied on the entire area to be irrigated before the crop on any part of the area reduces the moisture content of the soil to the permanent wilting point. The criteria to use as an index of when to start applying water will depend primarily upon the requirements of the crop itself as related to stage of development, presuming the irrigation system has been adequately designed. The most reliable index in current use is the moisture content of the soil. Experiments in Arkansas have shown that the yield of many of the crops grown is not reduced if the soil moisture content is allowed to be depleted to the 20 per cent available level.(5,6) However, for soils which restrict plant root development to the top one foot, there remains in the soil at 20 per cent available moisture only .20 to .30 inches of water available for plant use, or enough to sustain the

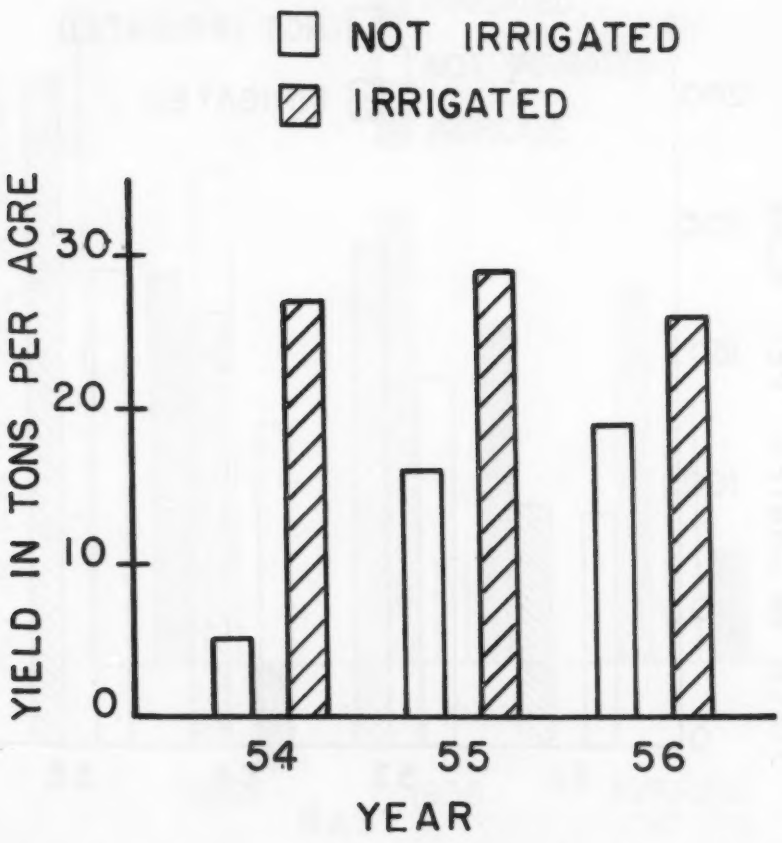

Fig. 3. Yield of forage sorghum.

plant for about one or two days. The experiments referred to previously also indicate that no decrease in yield results from beginning the application of water when the soil moisture content is depleted only to the 50 per cent available level at which point from 5 to 7 days water supply remains available to the plant. It is pointed out, also, that some crops produce optimum yields when the moisture content of the soil is maintained at above the 50 per cent available level.(7) From these experiments there has resulted the general recommendation that irrigation be started when the available moisture level in the soil is depleted to the 50 per cent level and that irrigation systems be designed to replenish the soil moisture supply before it is reduced to the wilting point.

The method of determining soil moisture content remains a serious problem for those advising farmers and for the farmers themselves who are irrigating. Devices utilizing electrical resistance and conductance principles

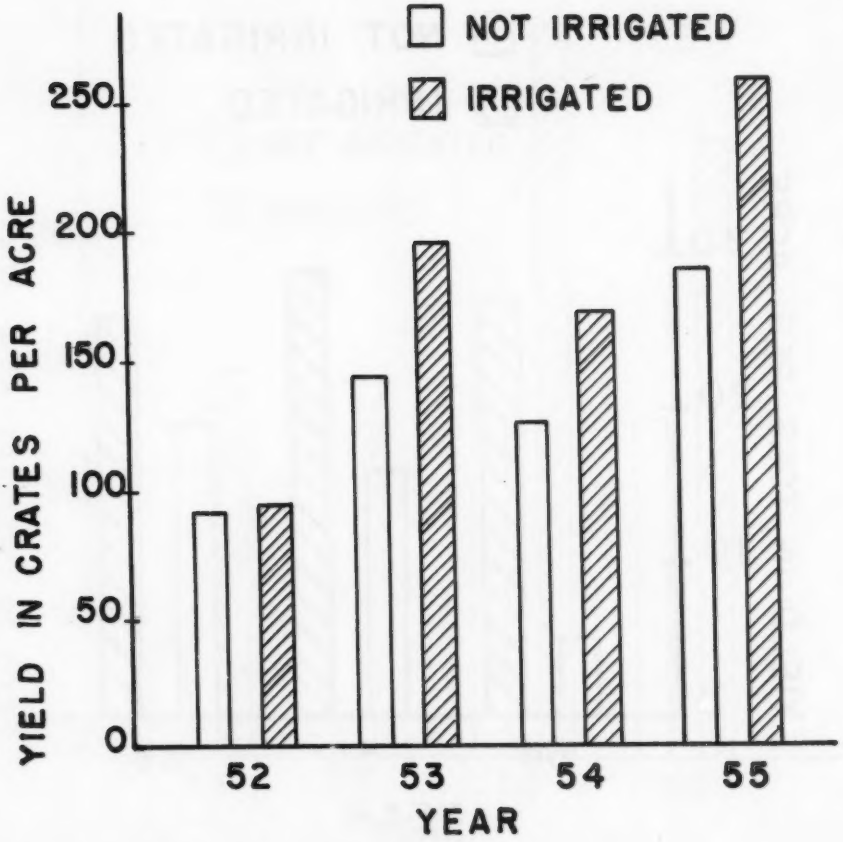

Fig. 4. Yield of Strawberries.

are available commercially but have been undependable. Conventional laboratory methods are not practical under most farming situations. The most reliable and practicable method of determining soil moisture continues to be by an examination and inspection of the soil in the root zone of the plants in several locations over the field. With some experience, one can attain some degree of accuracy in estimating soil moisture content.

It is important that each irrigation be made in sufficient quantity to replenish the moisture content of the soil to field capacity. The actual quantity of water to apply at each irrigation will depend upon the rooting habits of the plant itself, the water holding capacity of the soil and certain physical properties of the soil. The rooting habits of the plant and water holding capacity of soils are well known quantities and a discussion of them is beyond the scope of this paper.

It is important, however, to recognize certain physical characteristics of certain soils and adjust the quantity of water applied by irrigation to meet

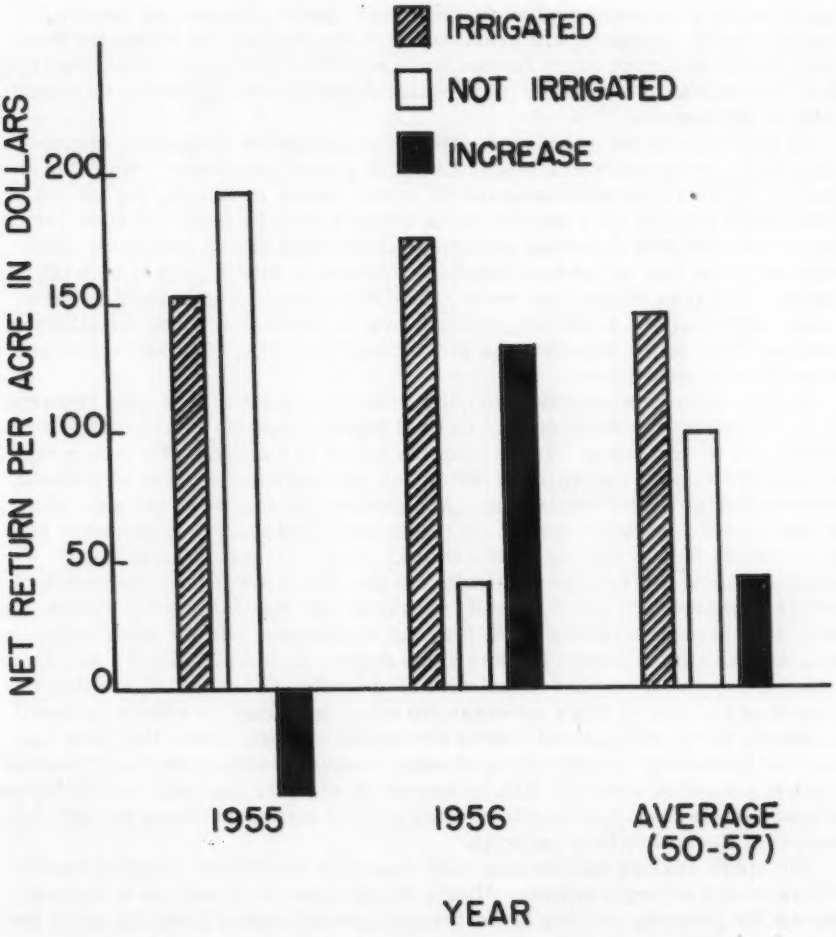

Fig. 5. Estimated net returns from cotton.

each situation. For example, many soils in the humid area have impervious layers beginning 9-12 inches below the soil surface which restrict the development of plant roots and limit the amount of water which can be applied at each irrigation. These impervious layers frequently are only a few inches thick but may extend to several feet. They are commonly known as pans and are both "man made" and genetic in origin. The "man made" pans are the result of implement traffic in the field and in some areas may be destroyed at intervals with mechanical equipment. In general, the genetic pans are the result of leaching and cannot be readily destroyed.

Engineers should encourage farmers who use gravity systems of irrigation to make more extensive use of water measuring devices in order to know how

much water is actually applied to each field. Weirs, flumes and meters, commercially available, are practical for farm use and are within the financial ability of almost every farmer who irrigates. Engineers designing irrigation systems should consider an adequate water measuring device an essential part of the system.

In determining the quantity of water to be applied by irrigation, engineers should advise farmers to disregard rainfall almost altogether. While weather forecasts are reasonably accurate for a large area, as of now, the amount of rain which will fall on a specific farm within a specific period of time cannot be determined with desirable accuracy. It is hoped that in the future more extensive use may be made of weather forecasting with regard to both the quantity and time of applying water. The Agricultural Engineering Department, University of Arkansas, has underway a project involving the utility of weather forecasting but since the project was started in 1958 no results are reportable at this time.

It can easily be shown that applying water in a quantity less than required to increase soil moisture content to field capacity results in almost no saving in the cost of irrigating. To this end the effect on the annual irrigation cost of pumping various quantities of water has been estimated. Data calculated for a sprinkler system delivering 1,000 gallons per minute to irrigate 100 acres of cotton show the cost of operating and maintaining the irrigation system to be $1.06 per hour or approximately $0.48 per acre-inch of water pumped. Since the average application in the humid area is approximately 2 inches of water, the cost of actually operating and maintaining the system for each acre is approximately $1.00 for each application. On the other hand, the total annual capital charge for this same system is calculated to be $18.31 per acre. It is evident that the actual cost of operating an irrigating system, exclusive of the cost of labor necessary to apply the water, is relatively small compared to the total annual cost of owning the system. Since the labor cost involved in making an application of water is approximately the same whether a small amount of water or a large amount of water is applied, very little cost benefit results from applying less water than is needed to return the moisture content of the soil to field capacity.

The above analogy applies also with regard to making an adequate number of irrigations during a season. After a farmer has purchased an irrigation system his primary concern is for the total annual cost of pumping water for irrigation. Estimates of the total annual cost of irrigating 100 acres of cotton with a sprinkler and gravity system have been calculated and are presented in Figure 6. The data indicate that as the quantity of water pumped on each acre increases, the total annual cost of pumping each unit quantity of water decreases. For example, the total annual cost of applying 4 inches of water is estimated to be $5.10 per inch for the sprinkler system. The cost of applying 6 inches of water, the eight-year seasonal average in Arkansas, is estimated to be $3.60 per inch.

It should be noted also that the total cost of irrigating is a relatively small percentage of the total cost associated with the production of cotton where irrigation is used. The data estimated for the conditions described elsewhere in this paper, see Table, indicate that all costs associated with irrigation, using a sprinkler system, represent only 11.3 per cent of all costs associated with producing cotton.

In assisting farmers in improvement of irrigation practices, engineers have the responsibility and unique opportunity to insure that irrigation systems

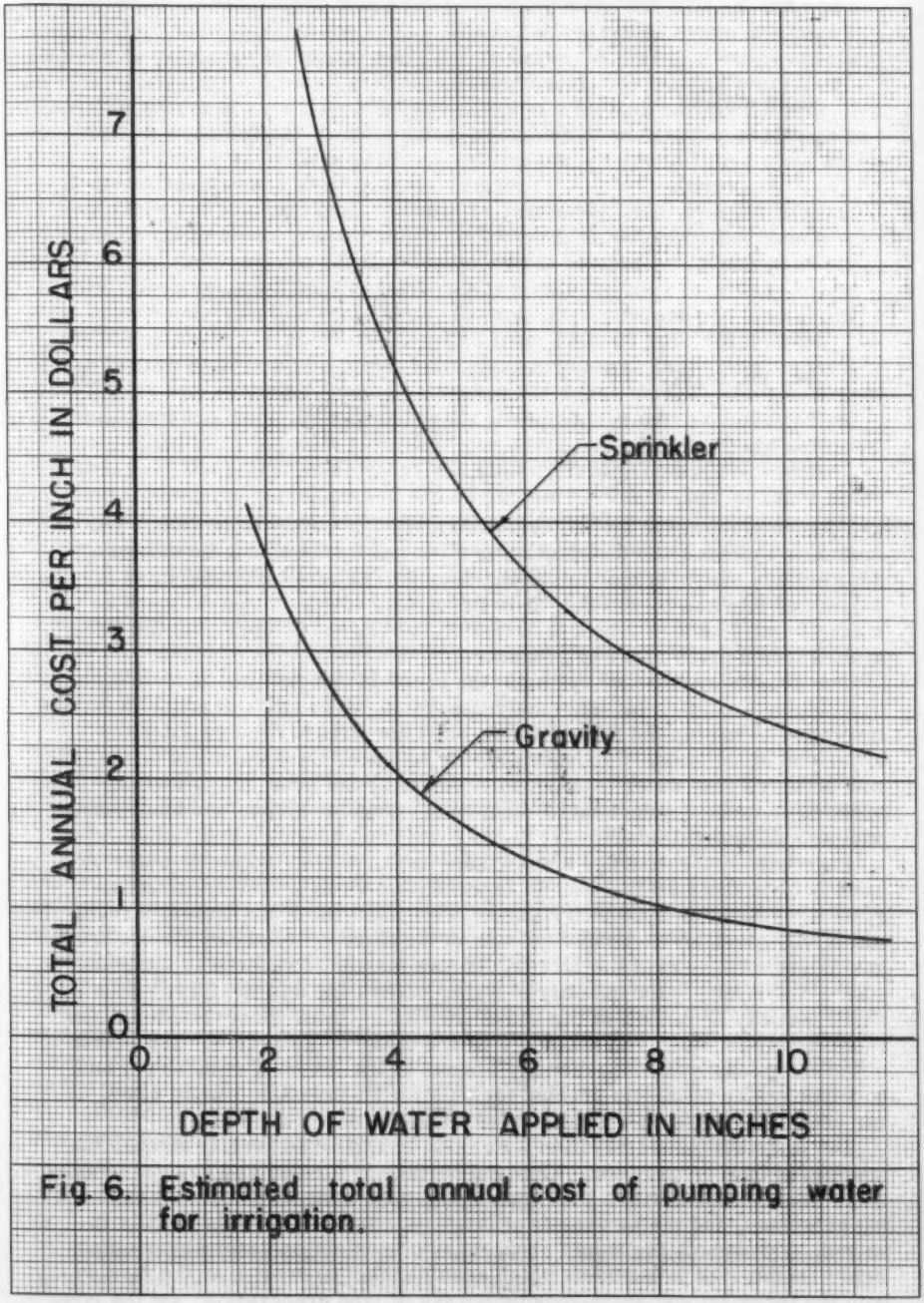

Fig. 6. Estimated total annual cost of pumping water for irrigation.

Table. ESTIMATED NET RETURN IN DOLLARS PER ACRE. 1950 - 57 INCL.

		SPRINKLER		GRAVITY
GROSS RETURNS		363.31		363.31
EXP:				
PRODUCTION	193.38		193.38	
IRRIGATION	24.65		11.45	
TOTAL	218.03		204.83	
NET RETURNS		145.29		158.48
INC. DUE GRAVITY				13.19
IRR. COST AS % TOTAL COST		11.3		5.6

are designed to insure the uniform distribution of water on the land. For sprinkler systems, the distribution is governed by the hydraulic design and its adequacy can be traced to the engineer who selected the equipment for the particular site and conditions. It is important, however, that farmers be made to understand that best distribution is obtained by operating the pump at or within a given range of pressures.

For gravity or flooding systems, the problem of obtaining uniform distribution is more complex but extremely important. A common practice in furrow irrigation is to stop the flow of water when the stream front reaches the lower end of the field. Consequently, the soil near the distant end of the furrow receives less water than does the soil near the point of application. The problem is further complicated because of irregular slopes and inadequate drainage on many fields.

Engineers can encourage a reasonably uniform distribution of water by employing in their design what is generally known as a "cut-back stream" system. Using this system irrigation is initiated using the largest stream which will cause no excessive erosion in the furrow. When the stream front has moved to the end of the furrow, its size should be reduced to the point where it will supply the quantity of water infiltrating into the furrow plus a small quantity of run-off at the end of the field. Under most conditions, the reduced stream should be about 1/3 to 1/5 of the original flow. The selection of the irrigating stream is directly related to the length of row, slope, and soil type on the field. No precise formula for its selection can be cited. It can best be selected by a trial of several streams on each field.

The distribution of irrigation water can be improved greatly through the medium of land preparation. The land surface is formed to a pre-determined slope in accordance with a pre-determined plan made by competent engineering personnel, and the movement of large quantities of earth is usually involved. Throughout the humid area, many competent engineers have established themselves in business which enables them to provide to the farmer both the technical and earth-moving phases of land preparation. On the large

farms where heavy equipment is available, farmers may find it more practical to employ an engineer to make the necessary maps and designs and use his own equipment to move the earth. The latter course of action has been given great emphasis in recent years due to the development of large wheel-type farm tractors and small scrapers having capacities ranging from four to eight cubic yards. An important advantage to a farmer moving the soil with his own equipment is that the actual operation of moving the earth can be done during times when the weather conditions permit and at times when other farm operations are not in progress.

Engineers and farmers need to know the investment that economically can be made in land surface preparation. To this end, an estimate has been made based on two assumptions: first, irrigation is to be used on 100 acres of cotton under physical conditions requiring a maximum or near maximum investment in water distribution equipment. Second, the net return per acre from irrigating cotton is the same as estimated from 1950-57 yield tests at the Cotton Branch Experiment Station, Marianna, Arkansas. In making the estimate furrow and sprinkler irrigation systems of equal capacity were designed. All equipment and services necessary for the sprinkler system required an estimated capital investment of $155.87 per acre. All equipment and services necessary for the furrow system, except land surface preparation, required an estimated capital investment of $70.65 per acre. These data in Table show that the estimated net return per acre using a sprinkler system, during the eight-year period, would be $145.29 per acre as compared to $158.48 per acre for a furrow irrigation system. This amounts to an average increase of $13.19 per acre in favor of the furrow irrigation system. If it is assumed that a reasonable rate of return on an investment is 6 per cent, the return of $13.19 per acre per year would indicate that a farmer can invest as much as $219.83 per acre in land surface preparation. This method of calculation can be used because land surface preparation from a legal point of view is an investment in the land and as such is not a depreciable item. It must be noted, however, that this value makes no allowance for any benefit that may be derived from improved drainage on the land. It should also be emphasized that this estimate of the amount which an individual farmer can afford to invest in land surface preparation is a maximum and is applicable to the specific area in and around the Cotton Branch Experiment Station where irrigation has caused an appreciable increase in yield over the eight-year period. Similar calculations can be made for conditions which have been encountered in other areas to arrive at an appropriate value for that specific locality.

Adequate drainage must be provided on any field which is to be irrigated. In the past it has been thought that drainage was primarily necessary on land which was irrigated using gravity methods only. However, many persons using sprinkler systems are finding drainage equally valuable. Experience has shown that frequently rainfall occurs immediately following an application of water which results in a supersaturated condition and the water which runs off collects in the low spots in the field with a resultant loss of production and considerable delay in operating cultivators through the field.

ACKNOWLEDGMENT

Acknowledgment is made to Dr. D. A. Brown, Dept. of Agronomy, Univ. of Arkansas for assistance in preparing the paper.

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Journal of the
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ANNUAL DROUGHTS[a]

M. C. Boyer,[1] F. ASCE

SUMMARY

The effects of annual droughts are reflected in the growth of tree rings, the formation of varves, records of temperature and precipitation, and the yield of crops. A study of tree rings has been helpful principally in determining variations in weather in arid regions. Varves were formed in lake deposits during interglacial times, and are not related to present-day climate. The records of temperature, precipitation and crop yield are the most useful tools in correlating droughts and crop yield.

A study of weather records and the yield of corn in Iowa is presented. It indicates that the gain in yield which irrigation would cause over a number of years might not warrant the additional expense.

Definition of a Drought

In succinct terms, a drought is a protracted dry spell. More specifically, for the purpose of this paper, it shall be considered as a period of lack of moisture during the crop-growing season, sufficiently long to damage or destroy a crop.

Types of Drought

There are two principal types of drought: (1) that which occurs in an arid region and results from an insufficient accumulation of snow and rain over high and distant reaches of a watershed; (2) that which results from inadequate or improperly distributed precipitation over the crops themselves.

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2153 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. IR 3, September, 1959.

a. Presented at meeting of Irrigation and Drainage Div., ASCE, Memphis, Tenn., September 25, 1958.

1. Hydr. Engr., Indiana Flood Control and Water Resources Comm., Indianapolis, Indiana.

The first is a characteristic of arid regions where crops are irrigated. It may build up over several years and become serious only when the ground water from shallow aquifers ceases to feed the stream, or when surface reservoir is exhausted.

The second is a characteristic of humid regions, and is manifested within a single season by the wilting or drying of the crop.

Inadequate precipitation is the principal culprit, though it is aided and abetted by dry winds and high temperatures, and cannot alone be held responsible. Since droughts in arid regions are often of several years' duration, the study of records of annual precipitation is usually sufficient to delineate them. In humid regions, on the other hand, inadequate or improperly distributed precipitation has a much more rapid effect, and the study of droughts requires examination of, at most, monthly records.

Historical Evidence of Drought

Any study of the potentialities of irrigation as a means of increasing crop yield must include an evaluation of historical evidence of drought. Principal sources of such information are records of tree growth, as evidenced by tree-ring characteristics; the sedimentation of interglacial lakes, shown by varves; and the records kept of stream flow, precipitation and crop yield. Each tells a story of the vagaries of climate, including the adequacy of moisture and the growth of vegetation.

Tree Rings

In 1901, Dr. A. E. Douglas, then teaching astronomy at the University of Arizona, applied a principle discovered by Theophrasties in 350 BC–that the annual growth rings of a tree are releated to the weather during the seasons of growth. Very recently, Kenneth McLeod, of Klamath Falls, Oregon, has applied this principle to a study of the water supply of the Klamath Basin.(1) He discovered many interesting facts concerning droughts in that region. The trees said that there had been a drought (apparently meaning a period of slow tree growth) from 1912 to 1952, now being followed by a period of good moisture supply, as also evidenced by an increase in the snow peaks on Mt. Shasta and the Cascade Peaks.

It should be pointed out that tree-ring studies have generally been made in arid regions of the American West, and are a measure of that type of drought, which involves long-range water supply problems and the requirement of water storage for irrigation.

Varves

While the banded clays of the lakes of interglacial time are of great interest to the glaciologist, they were laid down during a climatic period so different from the present as to be of no value in studies of recent droughts.

Records of Stream-flow and Weather

Records of stream flow, collected by the U. S. Geological Survey, and of weather, collected by the U. S. Weather Bureau, are of great assistance in evaluating the vagaries of climate and the potentialities of drought. The former provide data on the yield of watersheds, while the latter make possible

Table 1. Annual Precipitation and climatic conditions at Hays, Kansas, 1868-1957[1]/

Year	Precipitation, Inches Total	Average and climatic condition
1868	15.52	
1869	20.32	Drought period, 6
1870	19.79	years.
1871	23.00	
1872	16.36	
1873	20.68	19.28
1874	26.88	
1875	32.08	
1876	28.64	Mixed period, 11 years
1877	30.21	
1878	35.40	
1879	20.62	
1880	20.88	
1881	20.88	
1882	15.27	
1883	21.14	
1884	20.04	24.73
1885	25.95	
1886	28.74	Wet period, 5 years
1887	22.33	Floods in 1889
1888	20.18	
1889	27.36	24.91
1890	15.17	6 years,
1891	24.21	Drought period,/
1892	22.53	dust storms
1893	18.16	
1894	11.80	
1895	12.64	17.42
1896	27.70	
1897	26.73	9 years,
1898	31.55	Mixed period,/floods
1899	21.68	in 1903
1900	26.38	
1901	22.85	
1902	34.65	
1903	32.52	
1904	17.45	26.83
1905	23.73	
1906	23.08	Wet period, 5 years,
1907	25.40	floods in 1907
1908	25.33	
1909	28.27	25.16
1910	16.17	
1911	17.15	Drought period,
1912	20.20	5 years, crop fail-
1913	23.10	ures in 1914
1914	16.63	18.65
1915	34.14	
1916	16.01	
1917	16.92	Mixed period, 12 years
1918	23.58	
1919	26.13	
1920	21.28	
1921	18.46	
1922	18.98	
1923	27.10	
1924	14.51	
1925	23.02	
1926	16.45	21.38
1927	27.21	
1928	29.94	Wet period, 6 years,
1929	26.57	floods in 1928
1930	25.03	
1931	26.06	
1932	31.66	27.74
1933	16.26	
1934	16.06	Drought period, 5 years,
1935	21.87	dust storms
1936	15.90	
1937	17.86	17.29
1938	22.11	
1939	15.85	
1940	22.91	Mixed period
1941	28.13	
1942	29.61	
1943	16.19	
1944	29.70	
1945	20.34	23.11
1946	26.48	
1947	22.65	Wet period, 6 years,
1948	26.19	severe floods in 1951
1949	23.62	
1950	25.59	
1951	43.34	27.97
1952	13.39	
1953	21.07	Drought period, 1956
1954	18.44	lowest of record
1955	21.16	
1956	9.21	16.65
1957	28.33	

1. After data prepared by L. D. Wooster, Secretary, Western Kansas Development Association.

an evaluation of the climatic conditions in relation to the crops themselves.

Studies have been made of the variation in annual precipitation at Hayes, Kansas, by L. D. Wooster.(2) According to his analysis, the years can be definitely divided chronologically into groups according to the variation in annual precipitation. These he designates as "drought," "mixed," and "wet." He delineates an approximate 22-year cycle, composed of a 5-1/2 year drought, an 11-year "mixed" period, and a 5-1/2 "wet" period, which repeats in the same pattern. This record, shown in Table 1, covering 90 years, from 1868 to 1958, is very interesting.

Records of Crop Yield

The most significant factor in the study of the effects of climate (including drought) on crop production is annual crop yield. An analysis of crop yield and weather has been made by the U. S. Department of Agriculture.(3) These data have been studied, and the relations between crop yield, temperature and precipitation for the growth of corn in Iowa have been analyzed.

Fluctuations of Crop Yield in Humid Regions

The yields of crops in humid regions fluctuate with weather and with cultivation practices. Prior to about 1938, the variations in yield appear to have been caused primarily by the influence of weather. After 1938, the yields per harvested acre of such crops as tobacco in Kentucky, wheat in Kansas and corn in Iowa, showed decided increases. These are undoubtedly due to increasingly efficient farm practices, particularly the more intensive use of fertilizers and control of insect and plant diseases.

Analysis of Corn Yield in Iowa

Records of corn yield in Iowa, as given by the U. S. Department of Agriculture,(4) have been analysed in terms of two variables of weather: temperature and precipitation during the growing season.

Chronological Variations of Yield

In Figure 1 is shown corn yield, together with temperature and precipitation in chronological array from 1873 to 1948. The yield of corn varied from a minimum of 15 bushels per acre (buac) in 1894, when the precipitation during July was 0.63 inch and that for the growing season only 14 inches, to a maximum of 60 buac in 1948, due mainly to efficient farm practices. There does not appear to be a distinct trend from 1873 to 1938.

Variation of Temperature and Precipitation with Increasing Yield

The data were re-arranged in order of increasing yield and replotted (Figure 2). As indicated above, the minimum year was 1894, followed by 1936, 1934, 1890 and 1924, the 5 lowest years. The first 3 were characterized by high temperature and low rainfall, the following 2 by cool temperatures and somewhat subnormal rainfall. As the yield increases, neither the temperature nor the precipitation shows a marked tendency to either increase or decrease.

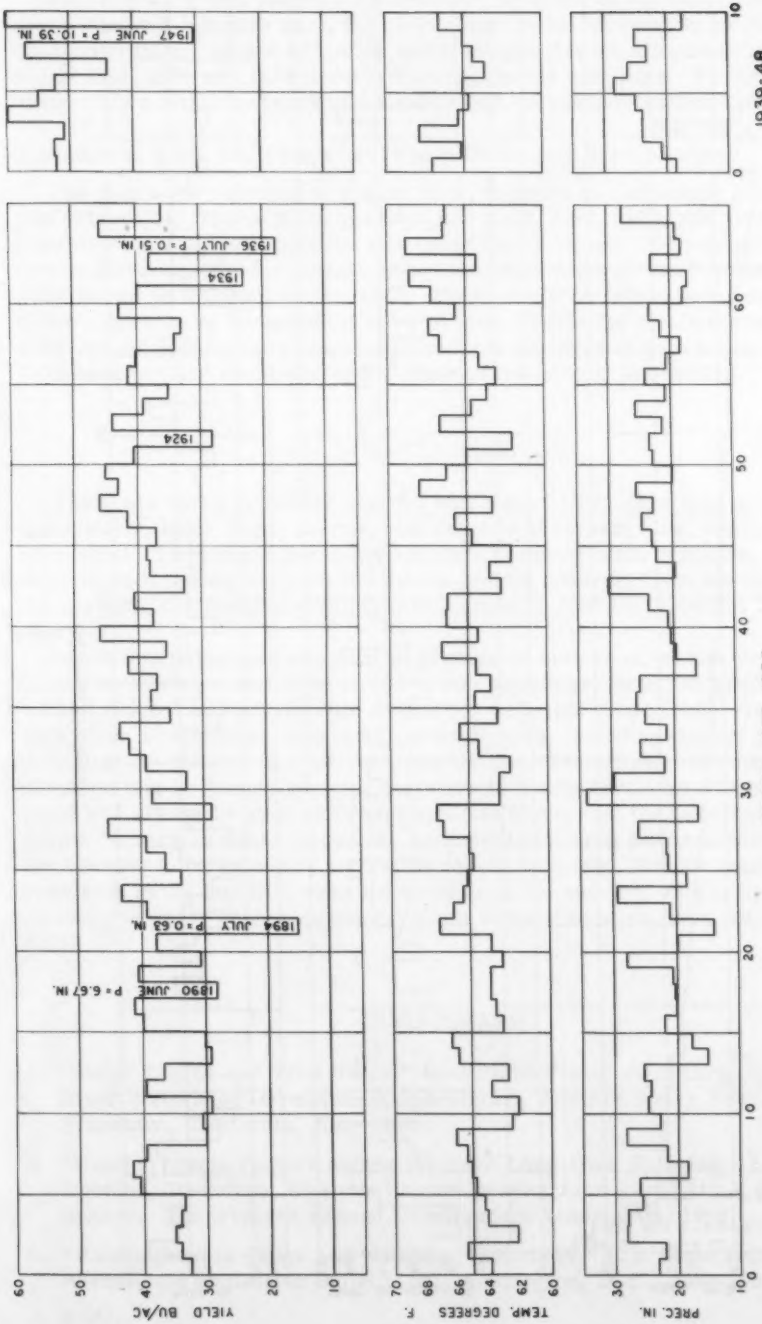

Fig. 1. Relation of corn yield, temperature, and precipitation in Iowa, 1873-1948. In chronological order

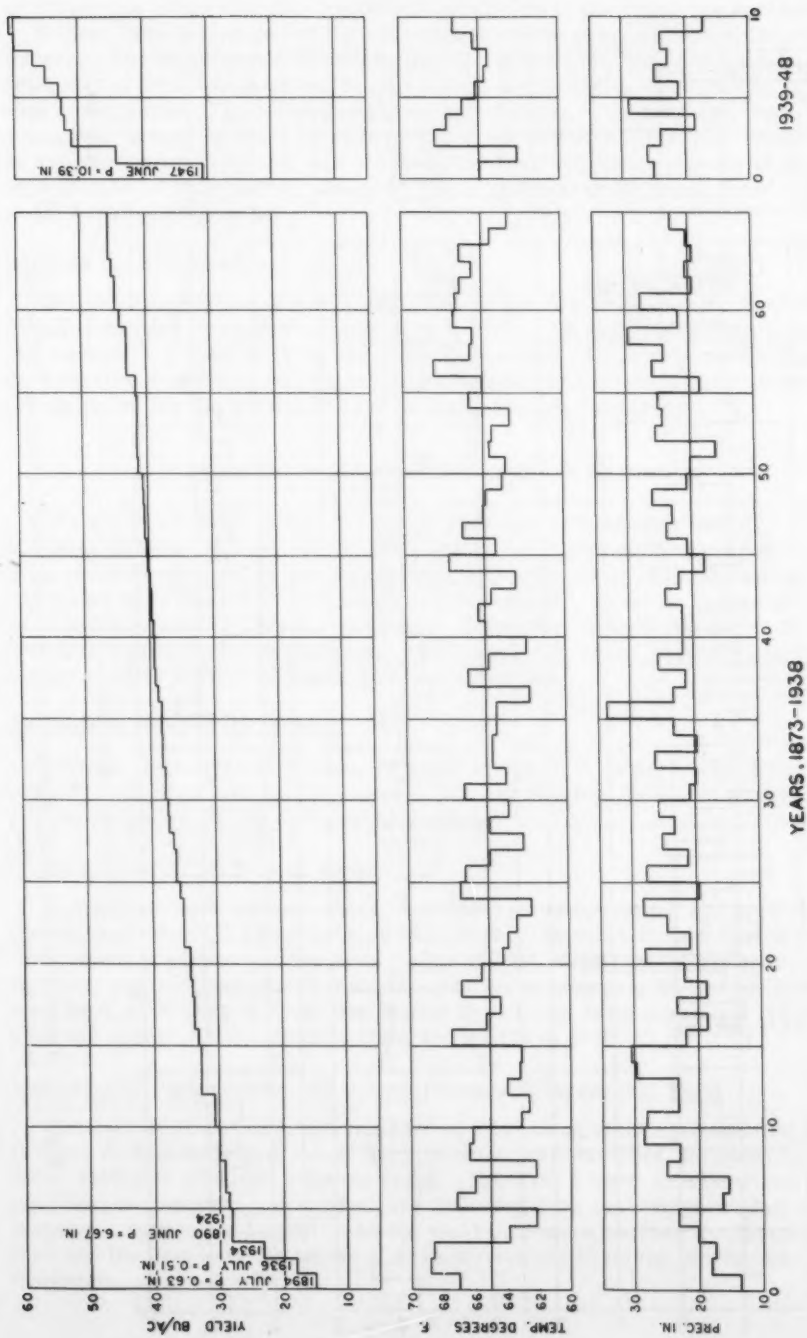

Fig. 2. Relation of corn yield, temperature, and precipitation in Iowa, 1873-1948. In order of increasing yield

Variation of Yield and Precipitation with Increasing Temperature

The data were rearranged a second time, in order of increasing temperature (Figure 3). In this case, the yield shows some tendency to increase until the temperature reaches 67°, after which the yield is erratic, and the low years 1894, 1934 and 1936 occurred during the hot summers. Precipitation shows only a slight tendency to decrease with increasing temperature.

Variation of Yield and Temperature with Increasing Precipitation

The data were rearranged a third time, in order of increasing precipitation (Figure 4). Three of the years of low yield, 1894, 1934, and 1936, occurred when the precipitation was less than 19 inches. It is important to notice, however, that for a range of precipitation from 20 to 33 inches, the yield shows no tendency to increase. There might be raised some doubt as to the efficiency of irrigation of corn in Iowa. While the yields during 1894, 1934 and 1936 might have been improved, the economy of gain which might have been brought about during the other years is open to question.

CONCLUSIONS

There are three principal ways by which annual droughts may be studied historically: tree rings; varves; and records of stream flow, weather and crop yield. Tree rings give a good picture of arid-region droughts, varves were formed during a period of climate greatly different from the present and not apropos, the records give the best picture, except that they are all too short.

While this paper was designed to present an historical picture of annual droughts, attention should be called to one significant item. In humid regions, lack of moisture is not the sole reason for deficient crop yields. Temperature plays a significant role, as do other factors, including time of planting, cropping and cultivation practices, and the use of fertilizer. It must not be assumed that by simply applying moisture to a crop through irrigation, its yield will always be so greatly increased as to warrant the additional expense. A very careful study of the past yield of a crop and an evaluation of the benefits to be gained by irrigation should be made. Not the least of the problems is the fact that when the weather is hot and dry, with irrigation warranted, the water supply, primarily from surface sources, may not be available.

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3. "Fluctuations in Crops and Weather, 1866-1948," U. S. Department of Agriculture Statistical Bulletin 101, Washington, D. C., June 1951.

4. Ibid.

Fig. 2. Relation of corn yield, temperature, and precipitation in Iowa, 1873-1948. In order of increasing yield

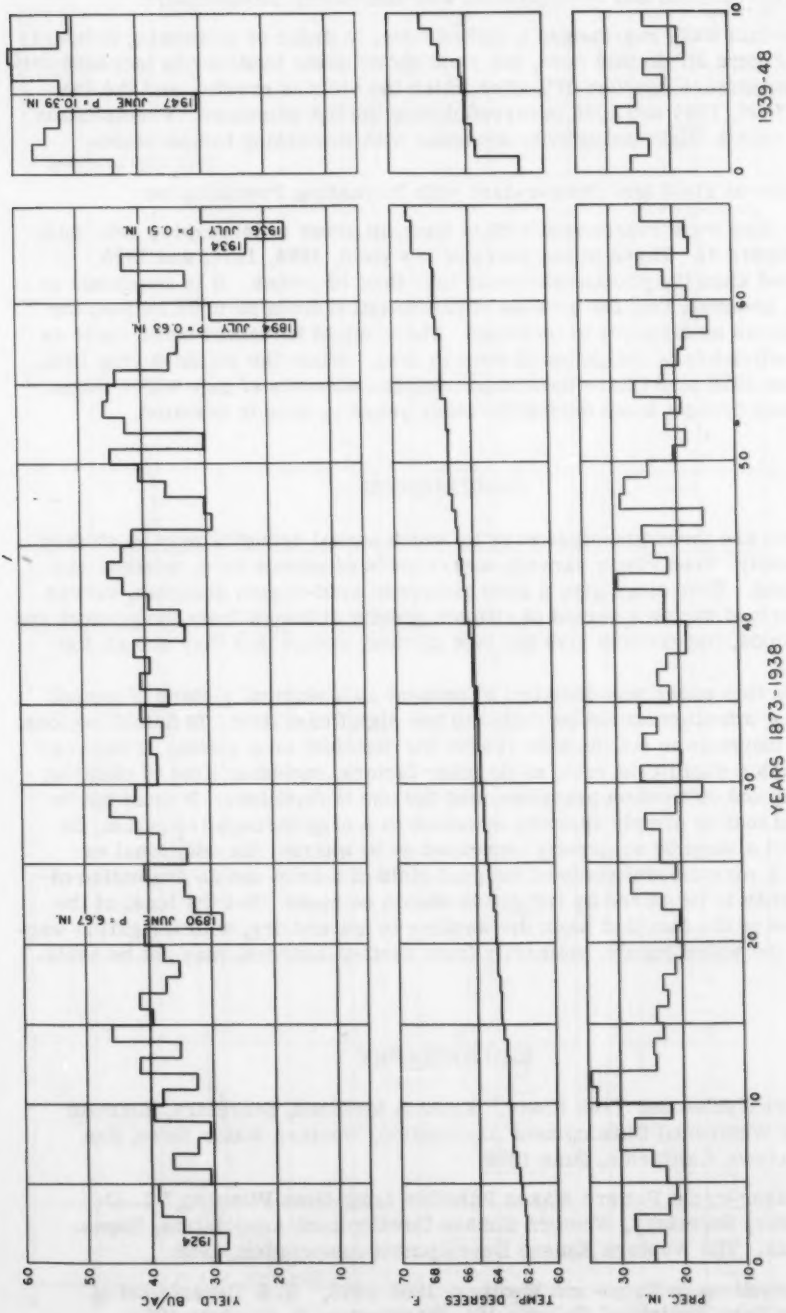

Fig. 3. Relation of corn yield, temperature, and precipitation in Iowa, 1873-1948. In order of increasing temperature

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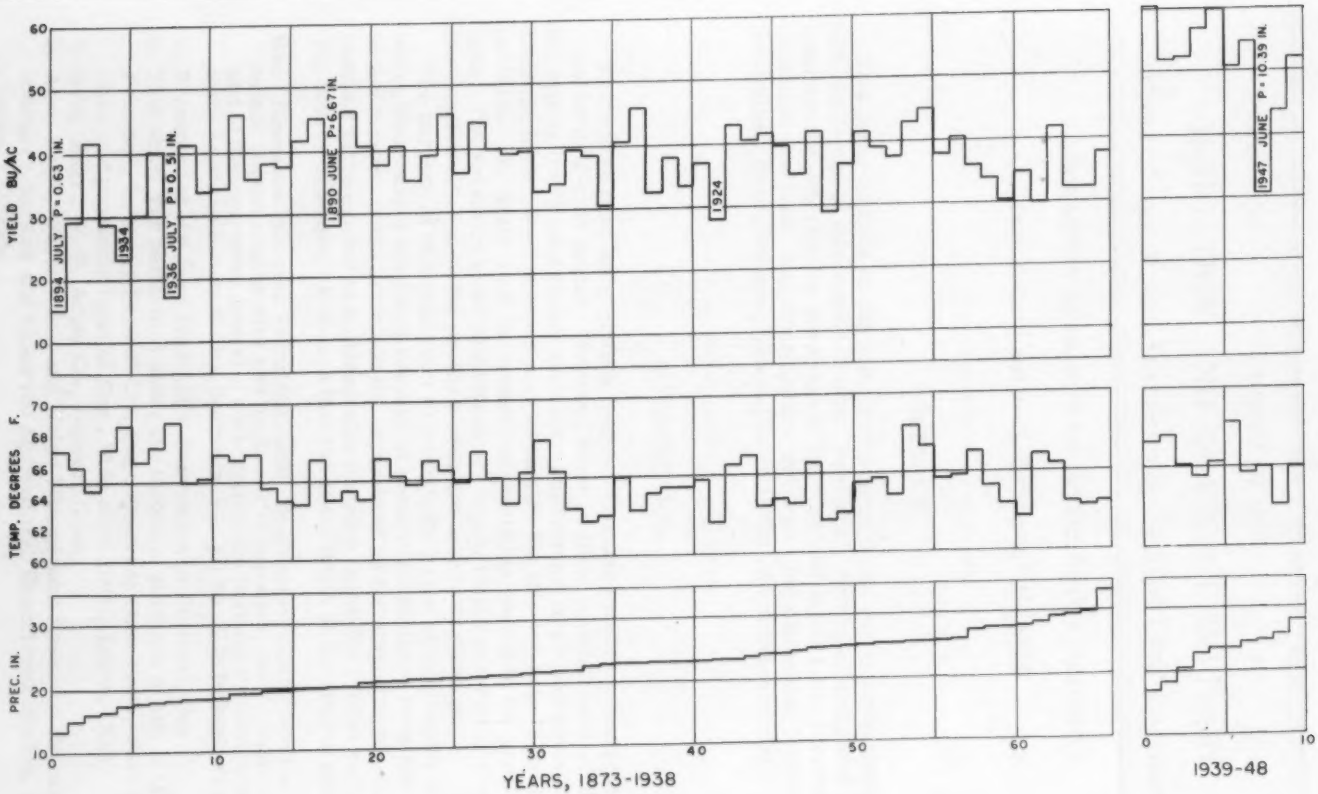

Fig. 4. Relation of corn yield, temperature, and precipitation in Iowa, 1873-1948. In order of increasing precipitation

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LABORATORY RESEARCH ON INTERCEPTOR DRAINS[a,b]

Jack Keller[1] and A. R. Robinson,[2] M. ASCE

ABSTRACT

This paper reports on the results of a large scale model study concerned with the design of interceptor drains. Previously developed analytical relationships for determining the shape of the resultant water table drawdown curve are analyzed. Relationships are developed for estimating the flow from the drains. Examples are given to illustrate the use of the material.

INTRODUCTION

There have been many studies made of drainage problems concerned with relief or grid-type drains. However, there is little information available on the hydraulics of interceptor drainage. Interception of ground water flowing laterally from a source which may be outside the affected area is the common problem. The drain must be located for maximum benefit to the problem area. The variables to be considered are depth, length and size of drain, and the location relative to the problem area and source of seepage.

The purpose of this study was to investigate a type of interceptor drain(2) where there was a source of seepage at some finite distance from the projected location of the drain. In the experiment an impermeable boundary with constant slope existed at a measurable distance below the ground surface. The source of seepage was such that the water depth at the source point would

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a. Presented at the June 1958 ASCE Convention in Portland, Ore.
b. This study was performed under the technical guidance of Dr. D. F. Peterson, formerly Head of Civ. Eng., Colorado State Univ., Fort Collins, Colo. and presently Dean of Eng., Utah State Univ., Logan, Utah.

1. Irrig. Engr., W. R. Ames Co., Denver, Colo.
2. Agri. Engr., Western Soil and Water Management Research Branch, Agri. Research Service and Colorado Agricultural Experiment Station, Fort Collins, Colorado.

remain unchanged after drainage. The factors which were investigated were the flow into the drain installation and the resulting drawdown curve.

The experiment was designed to establish the relationship between the pertinent variables and to obtain data for comparison with the results from other investigators. A check on the accuracy of theoretically derived relationships was one of the objectives. Dimensional analysis was used to relate the variables for a more systematic study.

Equipment

The study was conducted in the Hydraulics Laboratory at Colorado State University utilizing a large tilting flume which is shown schematically in Fig. 1. The flume was 70 feet long, 2 feet wide and 4 feet high and could be adjusted for slope from horizontal to 3 percent. The flume was filled with sand to a depth of 44 inches. A head and tail box with adjustable overflow devices were provided to control ground water levels. Tile drains were placed at three levels near the downstream end of the flume with an additional tile drain near the mid-point. Banks of manometers connected by plastic tubing to piezometers placed at intervals along the flume were used to determine the ground water profile. The outflow from the drains was weighed to determine the discharge.

The material used in the study was a decomposed granitic sand having a mean size of 0.107 inches (2.72 mm) and a uniformity coefficient of 2.0. Hydraulic conductivity was determined to be 0.038 feet per hour and a capillary rise of 1.5 inches was indicated. The material was compacted to uniform density as indicated by conductivity measurements made with variable depths of ground water in the flume. The porosity of the in-place material was determined to be 36.8 percent and the specific yield was 25.7 percent.

Procedure

The procedure used was such that the drawdown curve being investigated followed a higher drawdown curve. A minimum of three hours was allowed after a given set of boundary conditions was imposed in order that equilibrium be established. With the head water depth held constant, one of a series of drainage conditions was imposed either by opening the valve for any one of the file drains or by adjusting the level of the tail water. The tail box actually simulated an open, interceptor drainage ditch.

Head water depths were varied from 8 to 40 inches. Various tile drains were operated with each constant head water depth. The slope was varied in one-half percent increments from 0 to 3 percent.

Theoretical Analyses

Flow into Drain

At this point it is necessary to show by dimensional analysis the relationships that exist for the flow. For a simple system with no drain the variables for a two-dimensional system may be expressed by

$$q_o = \phi_1 (s , K , H) \qquad (1)$$

Fig. 1 Lay-out of tilting flume.

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q_o is the flow per unit width, K is the hydraulic conductivity and H is the depth of ground water above an impermeable boundary of slope s. Choosing K and H as repeating variables yields

$$\frac{q_o}{KH} = \phi_2 (s). \tag{2}$$

In general, one most determine ϕ_2 by experimentation. However, from Darcy's law

$$q_o = KHs \quad , \tag{3}$$

which leads to the conclusion that $\phi_2(s) = s$.

When a drain is installed as in Fig. 2, the relationship that exists is,

$$q_d = \phi_3 (s \ , \ K \ , \ r \ , \ L \ , \ H \ , \ h) \quad , \tag{4}$$

where q_d is the resulting flow in the drain, r is the radius of the tile, L is the distance from the tile line to the source of seepage, and h is the distance above the barrier layer to the drain. The remaining variables have the same meaning as in Eq. 1. Equation 4 may be rewritten in more usable terms:

$$q_d = \phi_4 \left[s \ , \ K \ , \ r \ , \ (H + sL) \ , \ H \ , \ h \right] . \tag{5}$$

Choosing K and H as repeating variables and combining, yields,

$$\frac{q_d}{HK} = \phi_5 \left[\frac{r}{H} \ , \ \frac{(H + sL)}{H} \ , \ \frac{h}{H} \ , \ s \right] . \tag{6}$$

One may infer from Eq. 6 that $q_d \neq q_o$ in Eq. 1 since additional parameters are involved. Dividing q_d/KH by the dimensionless parameter s yields,

$$\frac{q_d}{KHs} = \phi_6 \left[\frac{r}{H} \ , \ \frac{(H + sL)}{H} \ , \ \frac{h}{H} \ , \ s \right] . \tag{7}$$

Darcy's equation yields $KHs = q_o$, uniquely relating K and s. By substituting q_o for KHs, elimination of s as a separate variable appears possible, thus

$$\frac{q_d}{q_o} = \phi_7 \left[\frac{r}{H} \ , \ \frac{(H + sL)}{H} \ , \ \frac{h}{H} \right] . \tag{8}$$

The parameter r/H may be dropped since it is not important provided the tile is large enough to carry the required flow or an open drain is used. The converse of the reciprocal of (H + sL)/H was found to be more useful for showing the relationships, so Eq. 8 may be changed to,

$$\frac{q_d}{q_o} = \phi_8 \left[\frac{sL}{(H + sL)} \ , \ \frac{h}{H} \right] . \tag{9}$$

Equation 9 is the flow analysis of the type of interceptor drain studied.

Shape of the Drawdown Curve

An analysis similar to that for the flow analysis was made for the shape of the drawdown curve but was found to be difficult to handle because of the number of parameters involved. It was found that a previous theoretically derived relationship was applicable. One purpose of this study was to check this equation using model techniques. This equation which was developed by R. E. Glover and presented by Donnan(1) is,

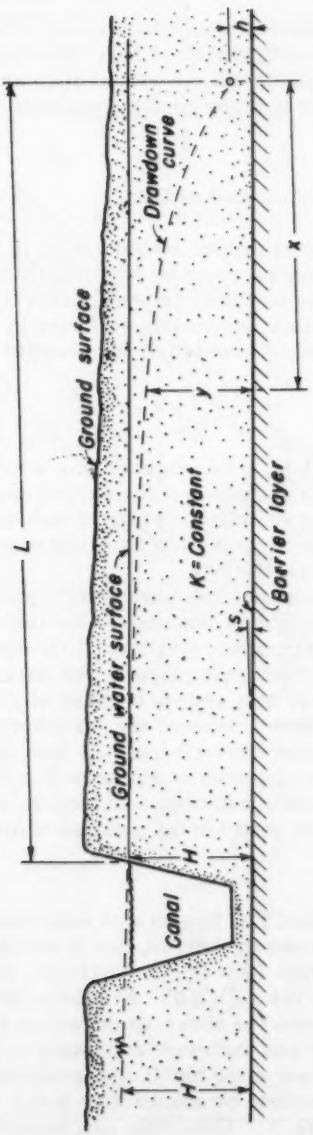

Fig. 2 Layout of a tile interceptor drain.

IR 3

$$x = \frac{H \log_e (H - h)/(H - y) - (y - h)}{s} , \qquad (10)$$

where x and y are the coordinates of any point on the drawdown curve as shown by Fig. 2 and the remaining variables have the same meaning as in Eqs. 1 and 4. This equation yields infinite values of x when s = 0 or y = H. The assumption was made that the flow remains constant before and after drainage.

Analyses of Data

If all variables had been given proper consideration in the dimensional analyses, the resulting parameters would be functionally related. The relationship would be demonstrated by the alignment of experimental data when plotted. The value of dimensional analysis is reflected by the degree to which the data are summarized, condensed or generalized by plotting in the dimensionless form.

Flow into Drain

In the analysis of the flow data, three assumptions were made. The first was that the capillary flow was negligible. The second assumption was that the tile drains were completely effective, i.e. there was no bypass flow over the drains. The third was that an open ditch interceptor was simulated by using the tail box to intercept all the flow.

Figure 3 is a plot of the dimensionless parameters given in Eq. 9. The parameter h/H is a ratio of height of drain above the impermeable boundary to the depth of water-bearing stratum. A value of h/H equal to zero indicated the drain was placed on the barrier layer. The parameter sL/(H + sL) shows the relationship of energy in a system because of slope to that due to slope and depth. This approaches a value of one for a great length or small values of H and decreases as the distance from the source to the drain decreases or H increases. The alignment of points in Fig. 3 indicates that all the factors influencing the problem had been considered. By using a dimensionless plot, the results can be used for the solution of problems of any size.

Shape of Drawdown Curve

The comparison of plottings of Eq. 10 and with experimental data is shown in Fig. 4. The results of two tests are shown, one of which could be considered a relatively short system (H = 40 , L = 811 , h = 0.0) and the other relatively long (H = 13.3 , L = 794 , h = 5.0). For the short system there was a considerable difference between the observed drawdown curve and the computed curve. Some adjustment was therefore necessary in Eq. 10. This was accomplished by computing a new value for H in the equation which will hereafter be called H' and was done by substituting x = L and y = H back into the equation and solving for H'. Using this new value for H', eq. 10 checked with observed data as can be seen in Fig. 4. Equation 10 then becomes,

$$x = \frac{H' \log_e (H' - h)/(H' - y) - (y - h)}{s} . \qquad (11)$$

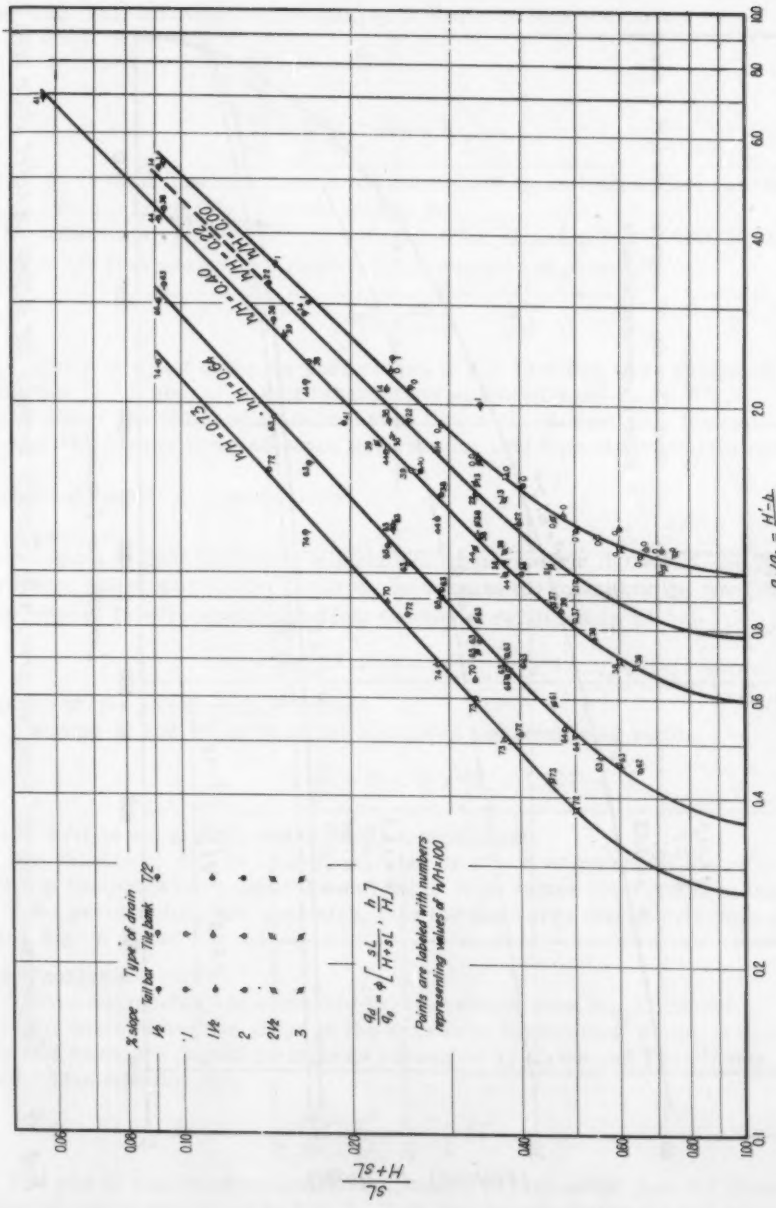

Fig. 3 Discharge of interceptor drains for slopes greater than zero.

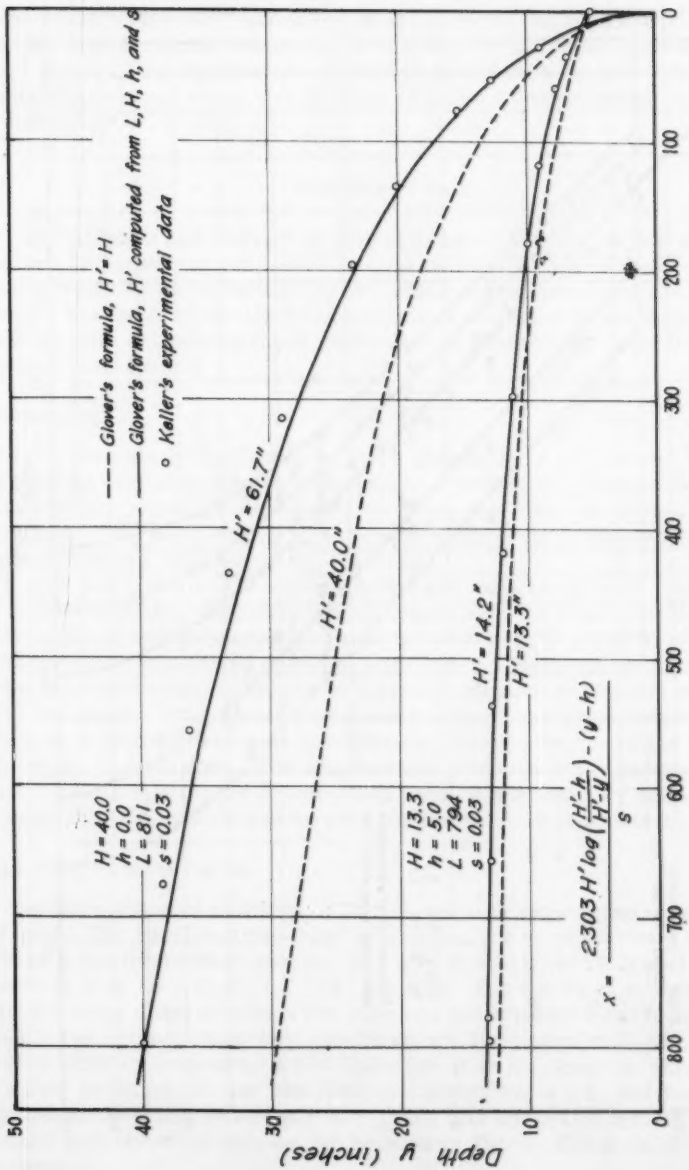

Fig. 4 Comparison of observed drawdown curves with Glover's formula for slopes greater than zero

At this point it was noted that the shape analysis was related to the flow analysis. The following relationship will illustrate this statement.

$$\frac{q_d}{q_o} = \frac{q_t - q_b}{q_o} \tag{12}$$

$$\frac{q_d}{q_o} = \frac{H'K_s - hK_s}{HK_s} = \frac{H' - h}{H} \quad , \tag{13}$$

where q_t is total flow in a system with drain and q_b is bypass flow past the drain. This relationship is shown in Fig. 3.

By substituting $(H' - h)/H$ for q_d/q_o , y for H and x for L into Eq. 9, which is the flow analysis, a related shape analysis is produced.

$$\frac{H' - h}{y} = \phi_9\left[\frac{sx}{y + sx} \quad , \quad \frac{h}{y}\right] \tag{14}$$

Figure 5 is a plot using the parameters in Eq. 14 which were computed using Eq. 11. It should be noted that this is essentially the same plot as Fig. 3 which was obtained from observed flow data. Essentially, one plot would give both the flow and shape analyses for this type of interceptor drain.

Horizontal Impervious Boundaries

Flow analysis

If a drain is installed above a horizontal impermeable layer, i.e. s = 0, the discharge intercepted by the drain q_d is equal to the total flow q_t since there is no bypass flow q_b and the general functional relationship is

$$q_d = \phi_{10}\ (K\ ,\ L\ ,\ H\ ,\ h\ ,\ r) \tag{15}$$

with the terms as previously defined.

Choosing K and H as repeating variables and combining yields,

$$\frac{q_d}{KH} = \phi_{11}\ \left(\frac{L}{H}\ ,\ \frac{h}{H}\right) \tag{16}$$

where r/H is again regarded as being insignificant.

The validity of Eq. 16 is demonstrated by Fig. 6 which shows the relationship computed from experimental data. With values determined in the field for permeability and geometry, then the discharge can be determined using Fig. 6.

Shape analysis

When a horizontal, impermeable barrier exists, then Eq. 11 cannot be used for determining the shape of the drawdown curve since s equals zero. For this case, the Dupuit formula as presented by Mavis and Tsui[3] was used. This equation is

$$\frac{H^2 - y^2}{H^2 - h^2} = \left(\frac{L - x}{L}\right)^n \tag{17}$$

The plot of this equation and corresponding experimental data for three different cases are shown in Fig. 7. This plot demonstrates the validity of the Dupuit equation for the case of zero slope.

Fig. 4 Comparison of observed drawdown curves with Glover's formula for slopes greater than zero

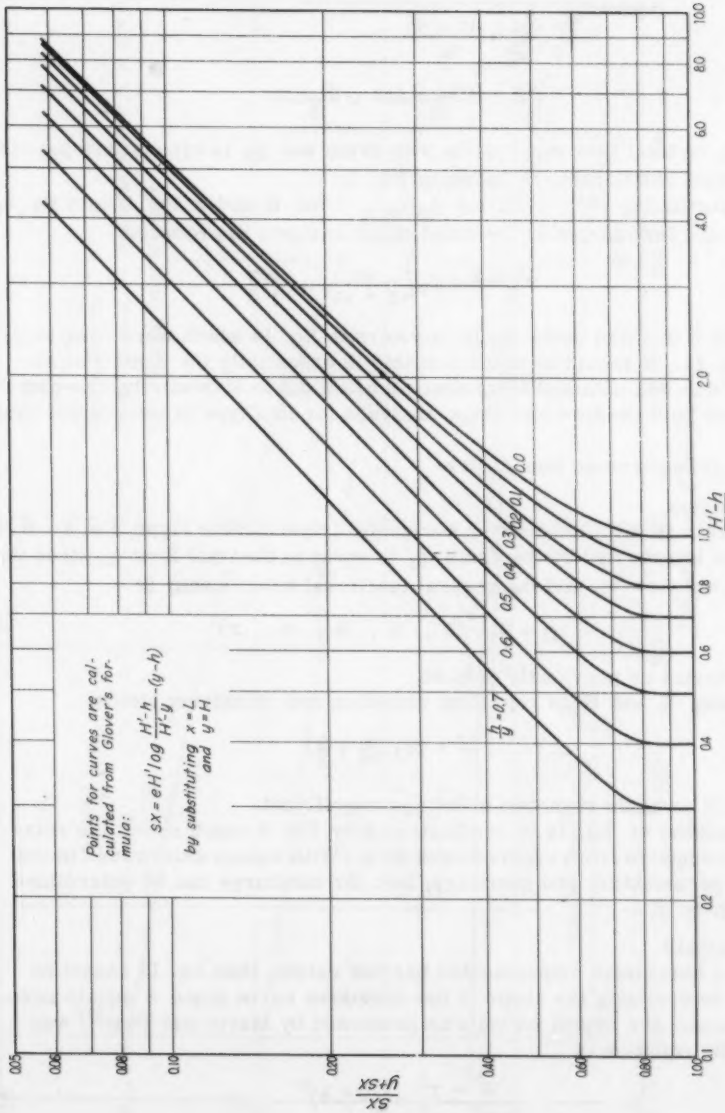

Fig. 5 Shape of drawdown curves for interceptor drains

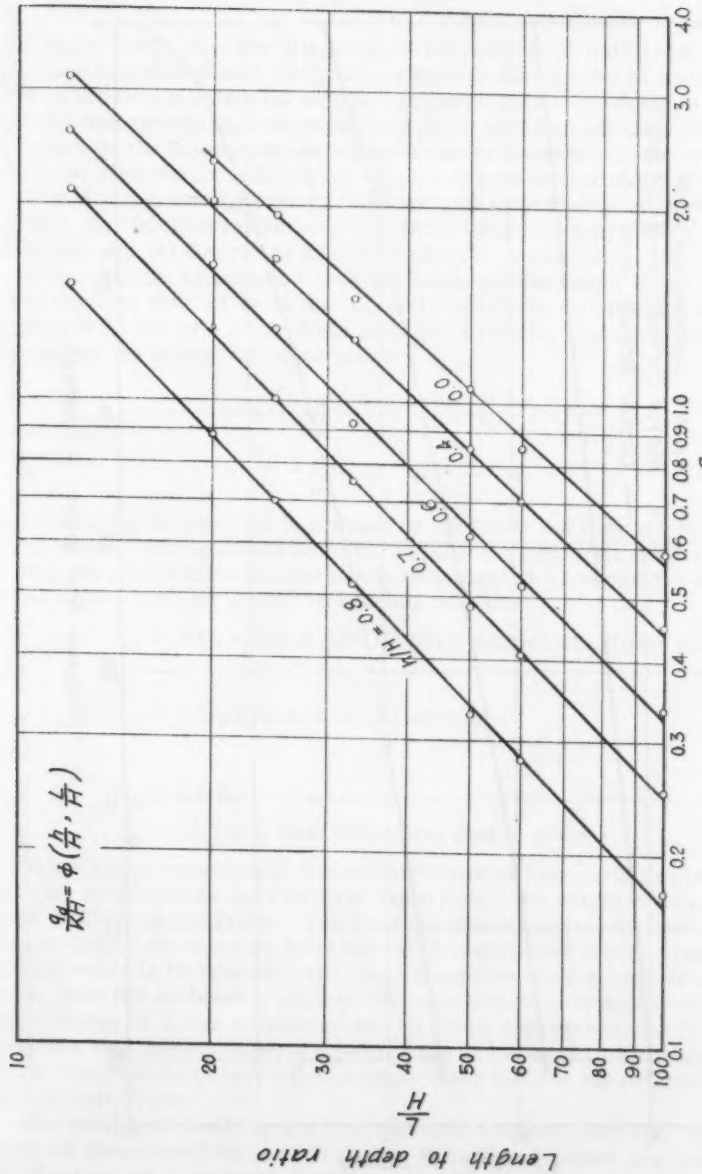

Fig. 6 Discharge as a function of distance to source for slope equal to zero.

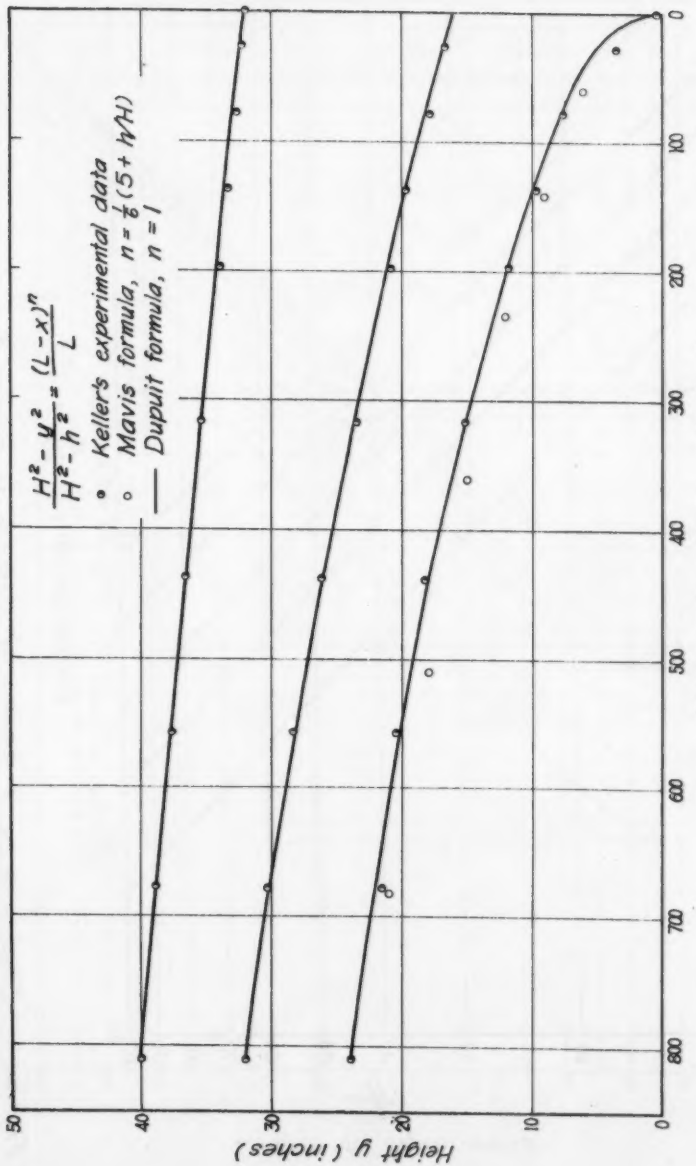

Fig. 7 Comparison of observed and calculated drawdown curves for slope equal to zero.

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Applications

Flow into Drain

In many cases, the flow which can be expected after a drain is installed is needed to properly design the drain. Figure 3 can be used to make an estimate of the flow if sufficient data are gathered before the drain is installed. For this determination, it is necessary to (1) make an estimate of distance (L) which is the length from the drain to the seepage source or to a point where the installation of the drain will not change the elevation of the water table to any appreciable extent; (2) determine thickness of the water-bearing aquifer; (3) find general slope of the water table or impermeable layer before drainage; and (4) determine average hydraulic conductivity.

As a practical example let it be assumed that the length (L) is 200 feet, the water-bearing stratum is 10 feet thick (H) overlying an impermeable boundary of slope 0.01 (s) with a tile drain installed 4 feet (h) above the barrier layer. Solving for the known variables yields

$$\frac{sL}{(H + sL)} = 0.17 \quad \text{and} \quad \frac{h}{H} = 0.4 \ .$$

From Fig. 3

$$\frac{q_d}{q_o} = 2.6 \quad \text{or} \quad q_d = 2.6\, q_o \ .$$

The discharge in the drain (q_d) would be 2.6 times the flow per linear foot which was occurring before drainage (q_o). If the hydraulic conductivity is known, the actual drain discharge can be computed. Assuming a conductivity (K) of 0.0001 foot per second (4.3 inches per hour)

$$q_o = HKs = (10)(0.0001)(0.01) = 0.00001 \text{ cfs/linear foot}$$

or

$$q_o = 4.5 \text{ gal/min/1000 linear feet}$$

since

$$q_d = 2.6\, q_o$$

$$q_d = 11.7 \text{ gal/min/1000 linear feet of drain}$$

In the foregoing example, the measurement of four variables is necessary in order to determine the resultant drain flow. The length L is probably the most difficult to determine. For the case of interception of canal seepage, this is simply the distance from the canal to proposed drain. When the ground water is flowing laterally from some undefined source at a great distance from the problem area, then the selection of L is also simplified. For large values of L the parameter $sL/(H + sL)$ approaches one (Fig. 3) so that the drain will merely remove a percentage of the total flow equal to the ratio of the installed depth below the original water table to the original depth of ground water flow.

The conditions under which interceptor drains are used may often be more complex than that of the simple steady-state flow systems just described and the selection of L is more a matter of judgment. Probably the most common problem is one in which the waterlogged condition is due to the operations of an uphill farmer who applies water at such times as his crops require. Each time water is applied a transient condition is initiated because of percolation

Fig. 7 Comparison of observed and calculated drawdown curves for slope equal to 0.0

to the water table. Since the solution presented is only for a steady state condition, it is necessary to adapt this method in order to estimate the flow which the drain will be required to handle.

The worst possible condition which the uphill irrigator could cause would be if the water table was maintained permanently at the ground surface at his field boundary. A steady state drain flow could then be determined, as in the foregoing example, using the distance from the proposed drain location to the uphill field boundary as L and the depth from the ground surface at the field boundary to the lower confining layer as H. The flow so determined would be larger than would actually occur except immediately after the drain was installed.

Shape of the Drawdown Curve

The shape of the drawdown curve resulting from the installation of an interceptor drain can be determined using Figs. 3 and 5. As a practical problem, suppose that the original depth of water-bearing strata (H) was 10 feet, the drain was 5 feet above the barrier layer (h) which had a slope (s) of 0.02 and was installed at a distance (L) of 500 feet from the known source of seepage. Then

$$\frac{h}{H} = 0.5 \qquad \frac{sL}{(H + sL)} = 0.5$$

from Fig. 3

$$\frac{(H' - h)}{H} = 0.69$$

$$H' = 11.9$$

The problem is to determine the distance (x) from the drain that the ground water would be lowered 2.5 feet from its original level. Then

$$y = 10.0 - 2.5 = 7.5 \text{ feet}$$

$$\frac{h}{y} = \frac{5}{7.5} = 0.67$$

$$\frac{(H' - h)}{y} = 0.92$$

from Fig. 5

$$\frac{sx}{(y + sx)} = 0.27$$

$$x = 139 \text{ feet}$$

Therefore, the ground water surface would be 2.5 feet below its level before drainage at a distance of 139 feet from the drain. Figure 5 can be used for finding the coordinates of any point on the drawdown curve.

SUMMARY

A method has been proposed for determining both the resulting flow and shape of the drawdown curve of an interceptor drain using dimensionless

plots. These plots were obtained from experimental data and previously determined theoretical relationships. This method is applicable for cases where a barrier layer is confining the flow through a relatively shallow strata and the source is either known or from engineering judgment an equivalent source is determined.

In many cases where a drain is constructed near the seepage source, such as a canal, the quantity of seepage may be increased to a large extent by the proximity of the drain.

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Definition of Terms

H — height of the original water table or head water depth above the impervious layer measured perpendicular to the slope (L).* (When small slopes are being considered, heights measured perpendicular to the slope or vertically are interchangeable.

H' — length characteristic used in Glover's formula to describe the drawdown curve, (L).

h — height of the water in the tile drain above the impervious layer measured perpendicular to the slope, or tailwater depth when a tile drain is not used (L).

K — hydraulic conductivity, a constant depending upon the properties of the porous medium and the water, (LT^{-1}).

L — upstream length of the interceptor drainage system, (L).

r — radius of the tile drain, (L).

s — slope of original water surface or slope of the impervious layer.

x — slope distance from the drain to a point on the drawdown curve, (L). (When small slopes are being considered horizontal and slope distances are interchangeable).

y — elevation of the drawdown curve above the impermeable layer measured perpendicular to the slope at a slope distance x from the drain, (L).

*Dimensional characteristics are indicated in parentheses.
L = length and T = time.

q_b discharge per unit width passing below the drain when the drain is placed above the impervious layer, i.e. $q_t - q_d$, (L^2T^{-1}).

q_d discharge per unit width intercepted by the drain or flow into the drain, i.e. $q_t - q_o$, (L^2T^{-1}).

q_o discharge per unit width from a system before the installation of an interceptor drain, (L^2T^{-1}).

q_t total discharge per unit width from a system with a drain, i.e. $q_d + q_b$ (L^2T^{-1}).

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STORAGE FOR IRRIGATION WATER IN HUMID AREAS[a]

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ABSTRACT

The factors that affect the development of irrigation storage facilities in humid climates are discussed. Also included are the modifications in design and planning criteria that are necessary in order to properly utilize the engineering knowledge gained on Western irrigation projects.

The irrigated acreage of the 31 Eastern States of the United States is now estimated to exceed 3,000,000 acres.(1) This is an increase of about 55 percent during the eight-year period, 1949-1956. This great expansion of the irrigated farmland in the eastern United States during recent years has introduced a new and important factor into the planning and development of water supplies.

The line that divides humid from arid lands in the United States is not a sharp one. It is a broad transition zone running across Texas, Oklahoma, Kansas, Nebraska, and North and South Dakota. East of this zone is the humid area which will be discussed in this paper.

In analyzing how irrigation water supply development affects water resources in this area, it should be kept in mind that the water which is used for irrigation is actually consumed, and only a small percentage can be reused for other purposes. This, plus the fact that the maximum irrigation requirements occur during the driest periods when rainfall is least and streams are lowest, certainly emphasizes the need for consideration of the potential irrigation water storage in planning the future development and use of our water resources.

The continued increase in irrigated acreage during both wet and dry years offers evidence that this is more than an "emergency" increase due to unusual

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weather conditions. Admittedly, much of the momentum that started irrigation farming off to such a rapid growth in the Eastern States was due to several severe drought years. However, farming practices and market conditions have so changed that many farmers can no longer afford the risks associated with unfavorable rainfall distribution in any year. Production costs of some crops have increased to such an extent that a further investment in irrigation is warranted to insure maximum high quality production every year.

The average annual rainfall in humid areas would be sufficient to meet the total moisture requirements of most crops if properly distributed throughout the growing season. This ideal rainfall pattern seldom occurs, however, and many crops suffer even in so-called "normal" years. Moreover, runoff from high-intensity storms and deep percolation to below the crop roots also deprive growing plants of a large part of the total rainfall.

The development of irrigation storage in the humid Eastern States up to the present time has been largely limited to the construction of reservoirs by individual farmers. A few group storage projects have been developed and a few more are in the planning stage but in general the constructed storage has been pretty much on an individual farm basis. The 1954 Census of Agriculture shows that 50% of all the farmers irrigating in the humid areas have some type of constructed storage. It is probable that as more and more farmers use irrigation, the unregulated flow of streams and natural lakes will become more fully utilized. As a result farmers will undoubtedly rely to an increasing extent on the use of constructed storage facilities.

If this is to be done in an efficient manner, it is essential that the larger group enterprise storage developments also be considered in order to provide the most economical projects.

Future years will undoubtedly see some large irrigation developments installed in the Eastern States, but in analyzing the time element involved in large project development, the following factors which differ from conditions in the Western States should be considered.

1. The economic feasibility of irrigating some farm crops has not yet been fully established. In planning larger projects which will include diversified farming areas such crops will normally be included and most be considered when computing benefits.

2. The increased profits over nonirrigated farming are much less than in the arid climates. This narrow profit margin further restricts the construction costs that can safely be charged against the land.

3. The rainfall and drought occurrence pattern will often dictate the time of irrigation. This will usually require that all farmers in the area be supplied with the maximum amount of irrigation water at the same time. This will tend to increase costs of the distribution system by requiring greater capacities per acre served.

4. Maintenance costs on canals and ditches will be increased due to the extended periods of non-use, and the more vigorous growth of weedy vegetation due to the higher rainfall. This could be offset with pipelines and lined canals, but again the installation costs would increase.

5. The smaller benefits from irrigation will require that a very high percentage of the land under the distribution system be included as a part of the project in order to provide a favorable cost-benefit ratio. Many farmers are

not yet ready for irrigation, which makes the formation of an operating organization such as an irrigation district or water users' association more difficult.

6. State laws must be modified to facilitate the formation of irrigation enterprise organizations, and to establish their legal rights to the water. The riparian doctrine which is now in effect in the Eastern States must be modified in order to protect the large investments that are being made in irrigation works.

Many of these factors do not apply, at least to as great a degree, to the individual farm type of irrigation development. Small watersheds will yield adequate supplies of irrigation water for individual farm acreages. Irrigation water can be used only on specialized crops that produce good returns; extensive distribution systems are not required; the individual farmer has complete control of the system and an operating organization is not needed. At the present time these factors tend to favor the individual type development, which in the long run may or may not be the most economical and efficient type of project.

It is evident that the pattern of growth of irrigation in the East is very similar to what occurred in Western United States. The first land to be irrigated out West was the land that could be supplied with water sheaply and easily. As economic conditions changed and the population increased, more complex and costly projects could be justified.

Similar progress is now evident in the Eastern States and at the present time they are in the first stage of water-supply development which consists of the direct diversions from streams, construction of wells, pumping plants, and storage reservoirs, primarily to serve the needs of the individual farmer.

Whatever the source of water, it must be able to deliver an irrigation stream large enough to cover the irrigated area in the allotted time, or conversely, the size of the irrigated area should be limited to what can be adequately irrigated by the available irrigation stream.

The rate of dependable flow required depends on the size of the area irrigated, the moisture requirements of the crops during periods of peak use, the type of soil in the area to be irrigated, the efficiency of the distribution system and the time allotted to complete one irrigation.

If the proposed water source will not supply irrigation water at a rate that will satisfy needs based on these factors, storage will be required. The amount of storage that is needed may vary from a small reservoir for overnight storage to a reservoir sufficiently large to supply the volume required for the entire irrigation season.

The primary factors that must be considered in determining the volume of storage required for irrigation purposes are: The water requirements of the crop, the farm irrigation efficiencies, the efficiency of the distribution system and the evaporation and seepage losses in the reservoir. The water requirements of the crop, which is the primary factor on which the need for irrigation storage is based, constitutes one of the primary differences between irrigation in the humid Eastern States and the arid areas of the West in that the volume of irrigation water required per irrigated acre is somewhat less. This is due primarily to the great differences in rainfall. Blaney[2] shows a typical irrigation water need of 42 inches for alfalfa in the Salt River Valley of Arizona while Dickinson[3] shows an average need of only 10 to 14 inches for forage crops in West Virginia, and Van Bavel[4] indicates that the

maximum amount of water needed for irrigation in North Carolina for nine out of ten years varies from an annual requirement of 8.3 inches to a maximum of 14.4 inches, when based on an irrigation application of two inches. These water needs reflect the actual needs of the plant, and of course, must be modified to allow for application efficiencies, evaporation, seepage, and transmission losses when planning for irrigation water storage requirements. Several procedures have been developed for estimating the water requirements of the crop on which irrigation storage volume must be based. Methods proposed by Blaney and Criddle,(5) Penman,(6) and Thornthwaite(7) have been used successfully for estimating potential evapotranspiration or consumptive use.

In the humid areas the "effective" rainfall will often provide a large percentage of this potential evapotranspiration or consumptive use, and therefore must be determined if any of these methods are used to estimate irrigation water storage requirements. This percentage of the rainfall that is effective for the production of farm crops will vary widely depending on the frequency, intensity and total amount of the rainfall occurrence. The method proposed by Van Bavel(4) which recognizes the available moisture-holding capacity of the soil within the root zone offers a procedure by which irrigation water requirements can be estimated directly, considering the probable frequency and amount of rainfall as it is related to the available moisture capacity of the soil and the rate of use by the crop. This procedure recognizes that the effect of short droughts on crops in the humid area depends in part on the moisture-holding capacity of the soil. A plant growing in a very sandy soil may show early signs of drought, while the same plant in a silt loam may survive a dry period without damage. These differences affect the need for irrigation water in each locality, and must be recognized in the determination of water needs. Van Bavel indicates a maximum seasonal water requirement of 17.6 acre-inches per acre in Eastern North Carolina for a soil requiring a 1-inch irrigation application while a soil that will hold an application of four inches has a seasonal requirement of only 10.4 acre-inches per acre.

Another important difference in irrigation water requirements is in the amount of water required for "leaching." The irrigation water used in the Eastern States is generally of high quality and in most cases in the coastal areas where brackish water is used the surplus rainfall seems to adequately handle the leaching of the harmful salts from the root zone. Reservoir capacity, therefore, would not normally be required to store water for leaching purposes.

After the actual irrigation water requirements of the crop have been determined, the farm irrigation efficiency must be estimated. Experience has shown that onfarm irrigation efficiencies in the Eastern States are slightly higher than the average figures quoted for the West based on the following reasons:

1. Higher percentage of sprinkler irrigation in the East. The more automatic nature of sprinklers, plus the simplicity of operation, enables the new irrigator to obtain higher efficiencies with less knowledge and effort and the widespread use of pipe distribution systems also decreases transmission losses. 78% of all farms now irrigating in the Eastern States now use sprinkler irrigation.

2. Surface irrigation is best adapted to the nearly level soils which have been properly smoothed. A high erosion hazard from rainfall is present on steep land. The low efficiencies that are obtained by surface irrigation on the steep lands of the West should not be a major factor in the East if proper provisions are taken to prevent soil erosion.

Land leveling for surface irrigation has continued a steady growth in the Eastern States. Farmers have found that land properly prepared for surface irrigation provides one of the most efficient surface drainage systems that can be installed on the farm. Land properly leveled facilitates efficient surface irrigation during dry periods and gives adequate surface drainage of surplus rainfall during wet periods.

3. Irrigation water is generally expensive. Nearly all irrigation systems have been installed during the period of higher costs since World War II, and in addition 94 percent of all farms being irrigated in the East use pumps to supply irrigation water. While total volume requirements are less the cost per acre-inch is often higher than in many areas of the West. This expensive water encourages the average Eastern irrigator to be efficient in water use.

4. Smaller applications generally required. The tendency of many crops to develop shallow root systems requires frequent small applications of irrigation water. This tends to reduce losses by overirrigation and deep percolation, recognizing that it also tends to increase the losses by evaporation from the soil surface. The humid area soils generally have fine-textured subsoils at 6 to 24 inches deep. The low permeability rates of these subsoils also tends to reduce losses by deep percolation.

Experience in irrigating does not appear to be a primary factor in producing favorable irrigation efficiencies as long as adequate technical assistance is available to the new irrigator. New irrigation farmers in the East seem to accept modern methods and techniques for improving irrigation efficiencies, thus partially offsetting their lack of experience.

When adapting farm irrigation efficiencies obtained in the Western States, these factors should be considered, recognizing, of course, that the design of each irrigation system should be based on the specific site conditions that are encountered.

The evaporation and seepage losses in the reservoir must, of course, also be estimated in the determination of the volume of irrigation storage that is required for any given project. Evaporation losses have been estimated using the methods outlined by Kohler, Nordenson and Fox[8] and Rohwer.[9] These methods have given good results for this purpose to date.

Seepage losses should be determined by basing the estimate on the conditions at a specific site. Complete geologic and soils investigations are necessary, and the results of local measurements and research work should be used whenever it is available. Smith[10] measured combined evaporation and seepage losses of 52.84 inches and 49.44 inches in 1953 and 1954 at McCredie, Missouri. Data of this type would be of great value in designing storage reservoirs where similar conditions are encountered. Additional measurements of seepage and evaporation under many varying conditions are urgently needed in order to more accurately determine these expected losses, especially as they apply to the smaller farm reservoir. Smith also points out the importance of steep side slopes and increased depths in the reservoir area as a factor in reducing evaporation and seepage losses. Plastics, butyl rubber, asphalt, soil

cement, and many other materials are now being utilized to reduce reservoir seepage losses. It should be emphasized that complete investigations prior to construction and the use of alternative reservoir sites can often eliminate the need for expensive reservoir linings.

One of the most difficult jobs in designing small irrigation storage reservoirs in the humid areas is the determination of the expected water yields from the watershed above the reservoir. Fortunately, some stream-gauging records are available, on which estimates of watershed yields can be based. Ogrosky(11) points out that there are approximately 2,962 stream-gauging records available for drainage areas less than 100 square miles. The Agricultural Research Service, U. S. Department of Agriculture, has obtained records on 328 watersheds of less than 10 square miles, with 294 of them less than one square mile in area. This data has been assembled by the Agricultural Research Service(12) and provides some useful data for estimating expected water yields from small watersheds.

The steady growth of irrigation in the Eastern States emphasizes the need for careful planning and good engineering in designing irrigation storage reservoirs. Additional research and investigations on various irrigation problems should be encouraged to insure that our limited water supplies are fully and efficiently utilized.

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AQUIFER TESTS IN THE SNAKE RIVER BASALT[a]

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ABSTRACT

The results of 11 aquifer tests and specific capacity data for 238 production wells indicate that the coefficient of transmissibility of the Snake River basalt ranges from 1×10^5 gpd per foot to 1.8×10^7 gpd per foot and averages about 4×10^6 gpd per foot. The coefficient of transmissibility of the entire thickness of the Snake River basalt probably greatly exceeds the values determined from test data because the wells for which data are available partially penetrate the aquifer. The coefficients of storage computed from test data are all in the water-table order of magnitude and range between 0.02 and 0.06. For the Snake River Plain, the average yield of the basalt to a 16-inch well during a well-acceptance test 8 hours in duration is about 2,100 gpm per foot. The aquifer ranks as one of the most productive in the United States. The average depth of well below land surface is 290 feet and the average penetration below the regional water table is 100 feet.

SUMMARY

The results of 11 aquifer tests and specific capacity data for 238 production wells indicate that the coefficient of transmissibility of the Snake River basalt ranges from 1×10^5 gpd per foot to 1.8×10^7 gpd per foot and averages about 4×10^6 gpd per foot. For the Snake River Plain, the average yield of the basalt to a 16-inch well is about 2,100 gpm per foot. Coefficients of storage range between 0.02 and 0.06.

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INTRODUCTION

The Snake River basalt is at or near the surface of and extends to considerable depths beneath most of the Snake River Plain, which extends more than 300 miles across southern Idaho. Through much of this vast plain the basalt ranges from a few hundred to possibly several thousand feet in thickness and consists of many interlocking lava flows spread out in successive sheets. The depth to the water table ranges from a few feet to more than 1,000 feet below the surface of the plain.

Each unbroken flow unit of the basalt is relatively impermeable, but porous, permeable zones along contacts between separate flows, joints, crevices and other voids yield and transmit large amounts of water to wells. Many individual wells penetrating 50 to 200 feet below the water table yield several cubic feet per second with less than a foot of drawdown.

Quantitative appraisal of the ground-water resources of the Snake River Plain requires the determination of the hydraulic properties of the basalt, the coefficients of transmissibility, and storage. One of the most important means available to the hydrologist for evaluating the hydraulic properties of an aquifer is the aquifer test, wherein the drawdown caused by pumping a well at one or more known constant rates is measured at the pumped well and at several observation wells tapping the aquifer. The formula most widely used to analyze aquifer-test data is the nonequilibrium formula of Theis (1935).

The characteristics of the water-bearing openings in the basalt are not exactly as assumed in the derivation of the nonequilibrium formula. The results of 11 aquifer tests presented later in this report, however, seem to indicate that, despite the differences between assumed and actual field conditions, meaningful values for hydraulic properties can be computed by applying the nonequilibrium formula with judgment to the data from aquifer tests in the Snake River basalt.

Nature of Water-Bearing Openings

Porous, permeable zones occur along many of the contacts between adjacent basalt flows. Later flows generally do not fill completely the voids in the irregular, brecciated, and fractured surface of earlier flows. The water-bearing openings of flow-contact zones are generally interconnected, although not uniformly so in all directions, and are distributed as widely as the flows they separate.

Tension joints caused by the shrinkage of the lava as it cooled and by differential movement of the hardened crust of thick basalt flows also provide reservoirs and conduits for the storage and transmission of ground water. The joints differ in size and at many places form intersecting sets. Systems of joints are irregularly distributed and interconnected.

Lava poured out into water solidify into pillowlike masses between which there is much open space. Such highly permeable pillow lavas occur at places in the Snake River basalt.

Beds of cinders are encountered in drilling in the basalt. The water-bearing characteristics of openings in the beds of cinders are similar to those of permeable sand and gravel. Cinder beds have high porosity and permeability and yield water to wells freely.

Lava tubes formed by outflow of liquid lava from beneath a hardened crust can transmit unusually large amounts of water and are often connected to other voids by joints and other fractures.

Vesicles produced by gas escaping from congealing lava and small pores between the mineral grains of the lava impart relatively high porosity to much of the Snake River basalt. The vesicles are poorly connected and the pores are so small that movement of water through them is greatly limited. However, these voids may be important to the storage capacity of the basalt.

Hydraulic Properties

The significant hydraulic properties of an aquifer are the coefficients of transmissibility, T, permeability, P, and storage, S. The field coefficient of permeability is defined as the rate of flow of water, in gallons per day, through a cross-sectional area of 1 square foot of the aquifer under a hydraulic gradient of 1 foot per foot at the prevailing temperature of the ground water. The coefficient of transmissibility is defined as the rate of flow of water, in gallons per day, through a vertical strip of the aquifer 1 foot wide and extending the full saturated thickness under a hydraulic gradient of 100 percent (1 foot per foot) at the prevailing temperature of the water. Thus, the coefficient of transmissibility is the field coefficient of permeability multiplied by the thickness of the aquifer, in feet. The coefficient of storage of an aquifer is defined as the volume of water it releases from or takes into storage per unit surface area of the aquifer per unit change in the component of head normal to that surface.

Permeability has little meaning in relation to an unbroken unit of the Snake River basalt. Permeable zones are concentrated along the contacts between separate flows. In addition, joints and other crevices are irregularly distributed throughout the basalt. The basalt, therefore, is heterogeneous and anisotropic (that is, its permeability varies greatly from place to place and with depth and direction of flow). The vertical permeability of the basalt is much smaller than the horizontal. Massive layers of relatively impermeable basalt and fine-grained interbeds often separate adjacent permeable zones. The hydraulic properties of any two successive zones are not likely to be the same. Wells in the Snake River basalt usually extend through 2 or more permeable interflow zones. During an aquifer test, adjustment of flow occurs between zones having different hydraulic properties.

Ground water occurs under confined (artesian) conditions in some parts of the Snake River Plain. For example, in the Mud Lake basin there are several flowing and nonflowing artesian wells. Over much of the plain, however, ground water occurs under complex water-table conditions which are not yet fully understood. Under water-table conditions water is derived from storage mainly by the gravity drainage of openings in the aquifer; however, the coefficient of storage includes not only the water drained by gravity from the dewatered volume (specific yield), but also a small amount of water derived from the underlying, still saturated zone by the compaction of the aquifer, and by expansion of the water itself.

The water table beneath the Snake River Plain occurs both in exceptionally porous materials (flow-contact zones, cinder beds, etc.), capable of yielding large quantities of water when allowed to drain, and in layers of massive basalt having a low specific yield. Thus, the cone of influence created by

pumping a well during an aquifer test ordinarily expands into materials having different storage properties during its growth. In addition, the water-bearing openings do not drain instantaneously as assumed in the Theis formula. For these reasons, the coefficient of storage may vary with time, especially during the early parts (the first few minutes or hours of pumping, according to aquifer conditions) of an aquifer test. As time passes, water rapidly derived from storage by the gravity drainage of relatively large voids is augmented by the very slow drainage of small pores (vesicles and pores between mineral grains) and crevices. Thus, the coefficient of storage computed from the results of a relatively short test (say 72 hours or less) may be much less than the true value.

Relatively impermeable layers of massive basalt and scattered layers of soil or water-laid sediments of low permeability occur both above and within the zone of saturation. These layers impede communication between the atmosphere and the water at different depths in the aquifer. As a result, ground water in the basalt behaves as though it were under artesian pressure, in that water levels in the wells respond to changes in atmospheric pressure. Also, the upper surface of the zone of saturation occurs in layers of relatively impervious basalt as well as in porous and permeable zones. Thus, the ground water is somewhat confined, preventing the rapid decay of changes in water levels caused by fluctuations in atmospheric pressure.

Aquifer-Test Theory

One of the best ways of analyzing aquifer-test data is by means of the nonequilibrium formula developed by Theis (1935). The nonequilibrium formula was developed on the basis of the following assumptions: that the aquifer is infinite in areal extent and is of the same thickness throughout; that it is homogeneous and isotropic; that it is confined between impermeable beds; that the coefficient of storage is constant; that water is released from storage instantaneously with a decline in head, and that the well has an infinitesimal diameter and penetrates the entire thickness of the formation. The ideal conditions assumed in the derivation of the nonequilibrium formula do not prevail in the Snake River basalt. Therefore, application of the nonequilibrium formula to the results of tests in basalt requires judgment based on knowledge of geologic conditions in order that meaningful values for hydraulic properties can be determined. It should be recognized that the computed results of a test are only approximations of average conditions in the vicinity of the test area.

Aquifer-Test Data

Aquifer tests in the Snake River basalt were made on wells at the National Reactor Testing Station (NRTS) near Arco, Idaho, in the Mud Lake basin, and in Minidoka County northwest of Rupert, Idaho. The results of two tests made at the NRTS are presented in detail to illustrate the analysis of data. The results of other tests are summarized later in this report.

An aquifer test (test 1) was made by the U. S. Geological Survey in cooperation with the U. S. Atomic Energy Commission on November 12-15, 1953, a group of wells (Fig. 1) in the northern part of the NRTS area was used. The generalized graphic logs of the wells are given in Figure 2. The effects of

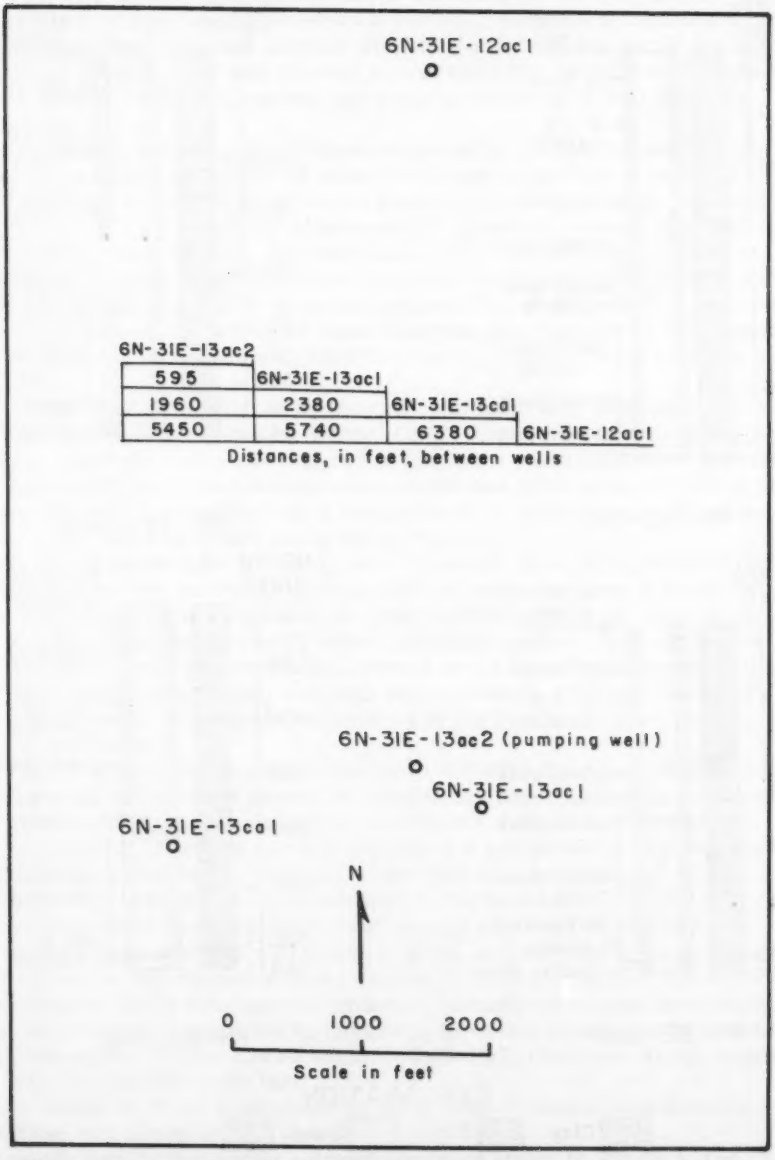

Fig.1 - MAP SHOWING LOCATIONS OF WELLS USED IN TEST 1.

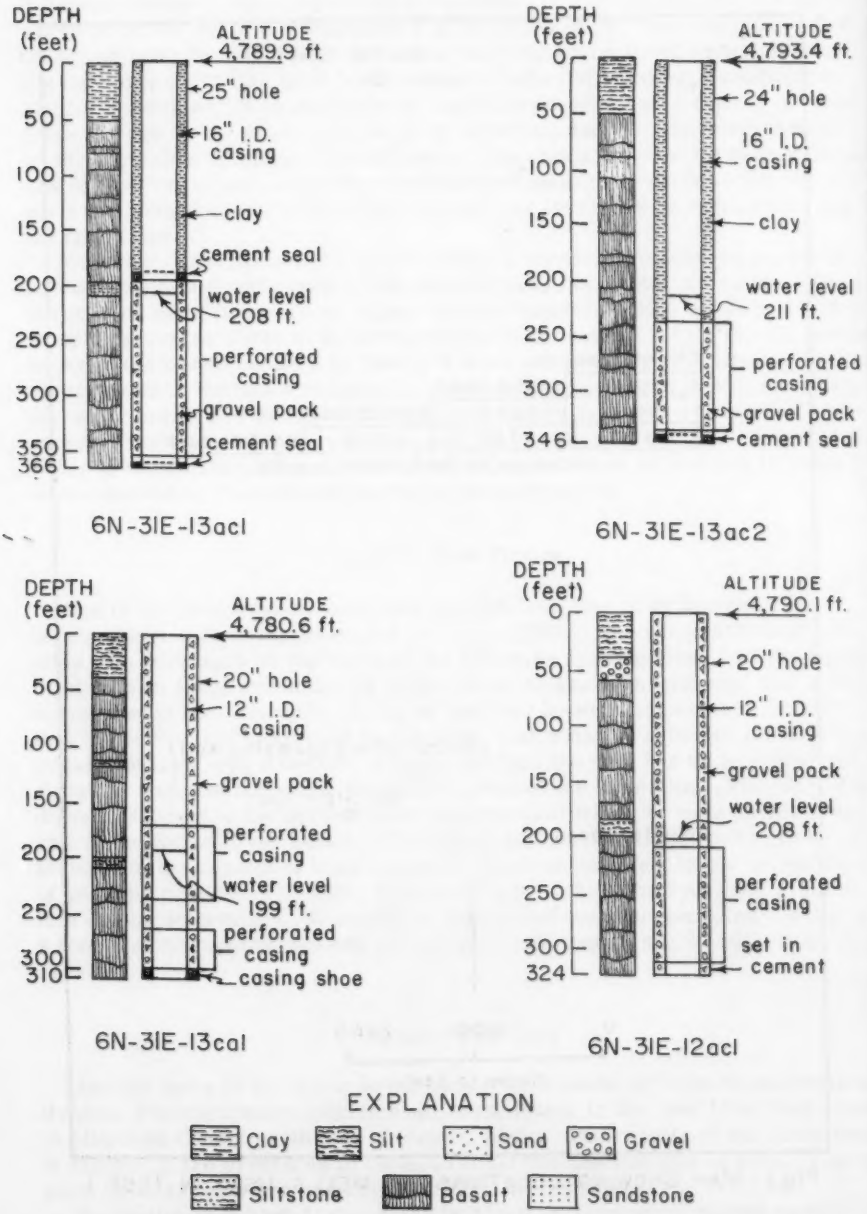

Fig. 2 - GENERALIZED GRAPHIC LOGS OF WELLS USED IN TEST 1.

pumping well 6N-31E-13ac2 were measured in observation wells 6N-31E-13ac1, 6N-31E-13ca1, and 6N-31E-12ac1. Pumping was started at 12:30 p.m. on November 12 and was continued for a period of 72 hours at a constant rate of 1,220 gpm. Pumping was stopped at 12:30 p.m. on November 15 and the water level in the aquifer was allowed to recover. Figure 3 shows the effect on water levels caused by starting and stopping the pump in well 6N-31E-13ac2.

Drawdowns in the pumped and observation wells are determined by comparing the extrapolated graphs of water levels measured before pumping started with the graphs of water levels measured during pumping. The drawdown data were then adjusted for atmospheric-pressure changes that occurred during the test (see Fig. 3). The barometric efficiencies of the wells were determined by using water-level data collected during the latter part of the test. Adjusted drawdowns were plotted against time on logarithmic or semilogarithmic paper. The adjusted time-drawdown data based on a barometric efficiency of 88 percent for well 6N-31E-13ac1 are given, as an example, in Figure 4.

The test data were analyzed by the graphical method of superposition devised by Theis and described by Wenzel (1942) to solve the nonequilibrium formula. The type curve was matched to the latter part of the time-drawdown data, and match-point coordinates were substituted in the nonequilibrium formula to compute the coefficients of transmissibility and storage. Computations for well 6N-31E-13ac1 are given in Figure 4.

The drawdown data for the early part of the test (first 90 minutes of pumping) are interpreted as departing from the type curve because of the inability of the basalt aquifer to respond to pumping instantaneously as would be homogeneous, isotropic, and perfectly elastic artesian aquifer. With prolonged pumping, adjustment of flow occurs between zones of different permeability, the effects of delay in gravity drainage become small, and the differences between the water-bearing characteristics of the basalt and ideal conditions become negligible.

Adjusted drawdowns in observation wells 6N-31E-13ac1 and 6N-31E-13ca1 at the end of the test were plotted on logarithmic paper against the squares of the distances from the respective observation wells to the pumped well, to yield a distance-drawdown curve (a portion of a profile of the cone of influence). The type curve was matched to the distance-drawdown curve and match-point coordinates were substituted in the formula for computation of the coefficients of transmissibility and storage as shown in Figure 5.

The drawdown in well 6N-31E-12ac1, which is 5,450 feet from the pumped well, cannot be determined with any degree of accuracy because the water-level changes due to atmospheric-pressure fluctuations almost completely mask water-level changes due to pumping. However, careful study of water-level data shows that the drawdown in well 6N-31E-12ac1 was in the magnitude of 0.1 foot at the end of the test.

The values of T and S computed by using time-drawdown and distance-drawdown data are given in Table 1.

As indicated by the distance-drawdown curve shown in Figure 5, the 72-hour test sampled an area of basalt having a diameter of roughly 8 miles. The coefficients computed from the results of the test represent the average hydraulic properties of the basalt within that large cone of influence. Considering the complexity of ground-water conditions in the basalt, the agreement between the values of T and S computed from the results of the test is

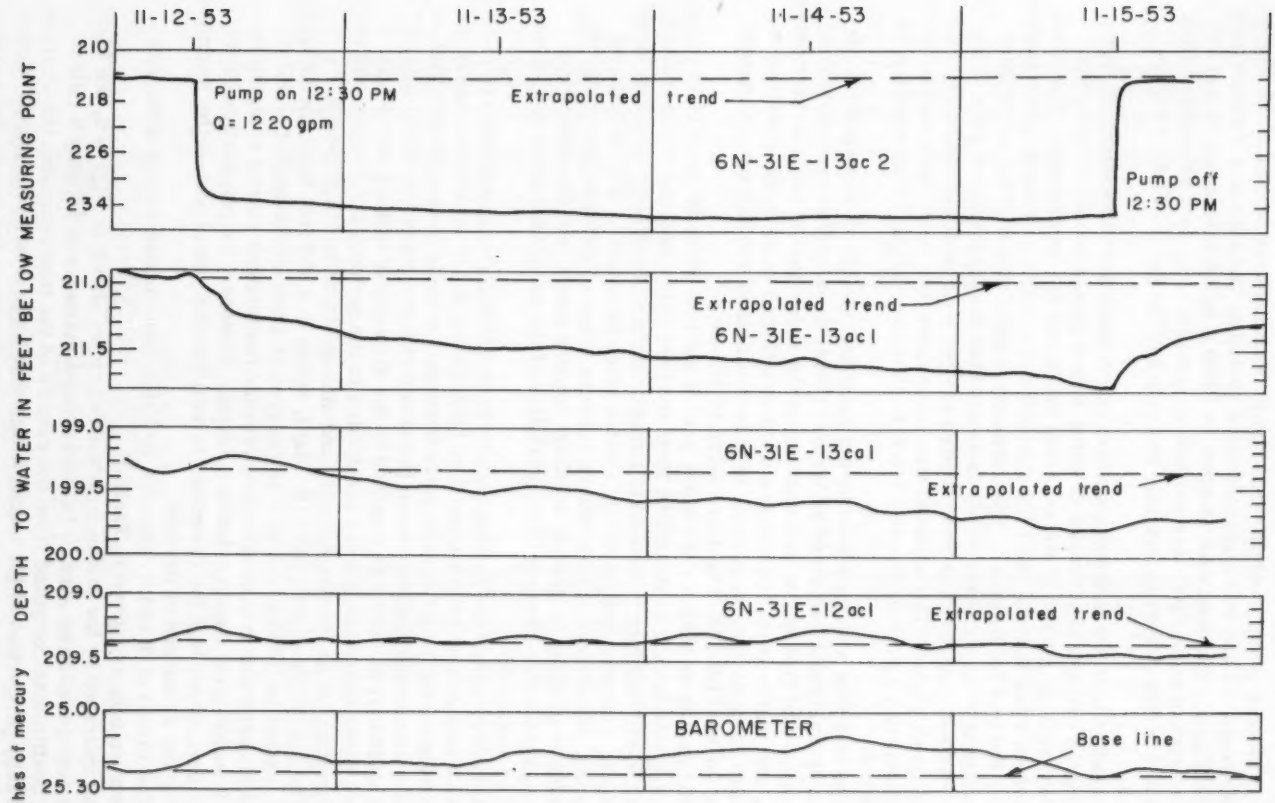

Fig. 3 - EFFECT ON WATER LEVELS CAUSED BY STARTING OR STOPPING PUMP IN WELL 6N-31E-13ac2

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Fig. 3 - EFFECT ON WATER LEVELS CAUSED BY STARTING OR STOPPING PUMP IN WELL 6N-31E-13ac2

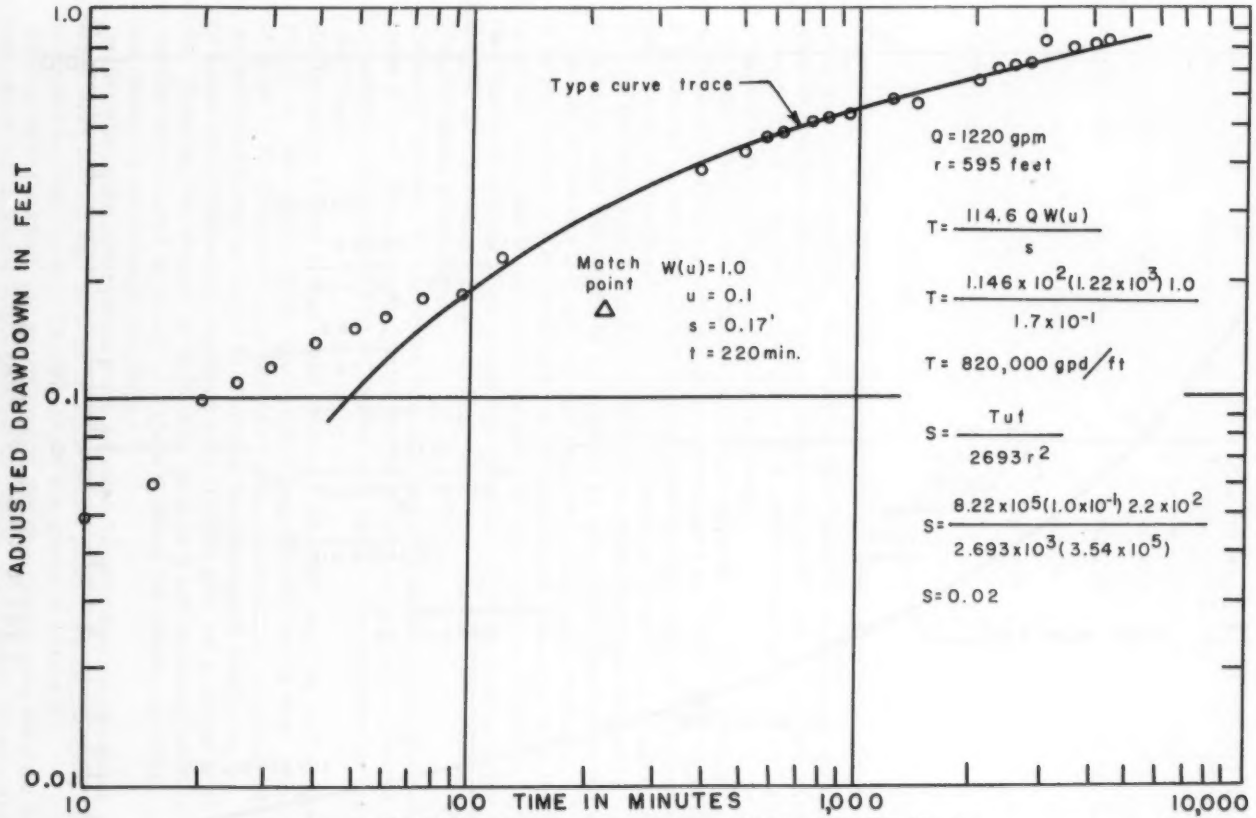

Fig. 4 - TIME-DRAWDOWN GRAPH FOR WELL 6N-31E-13ac1.

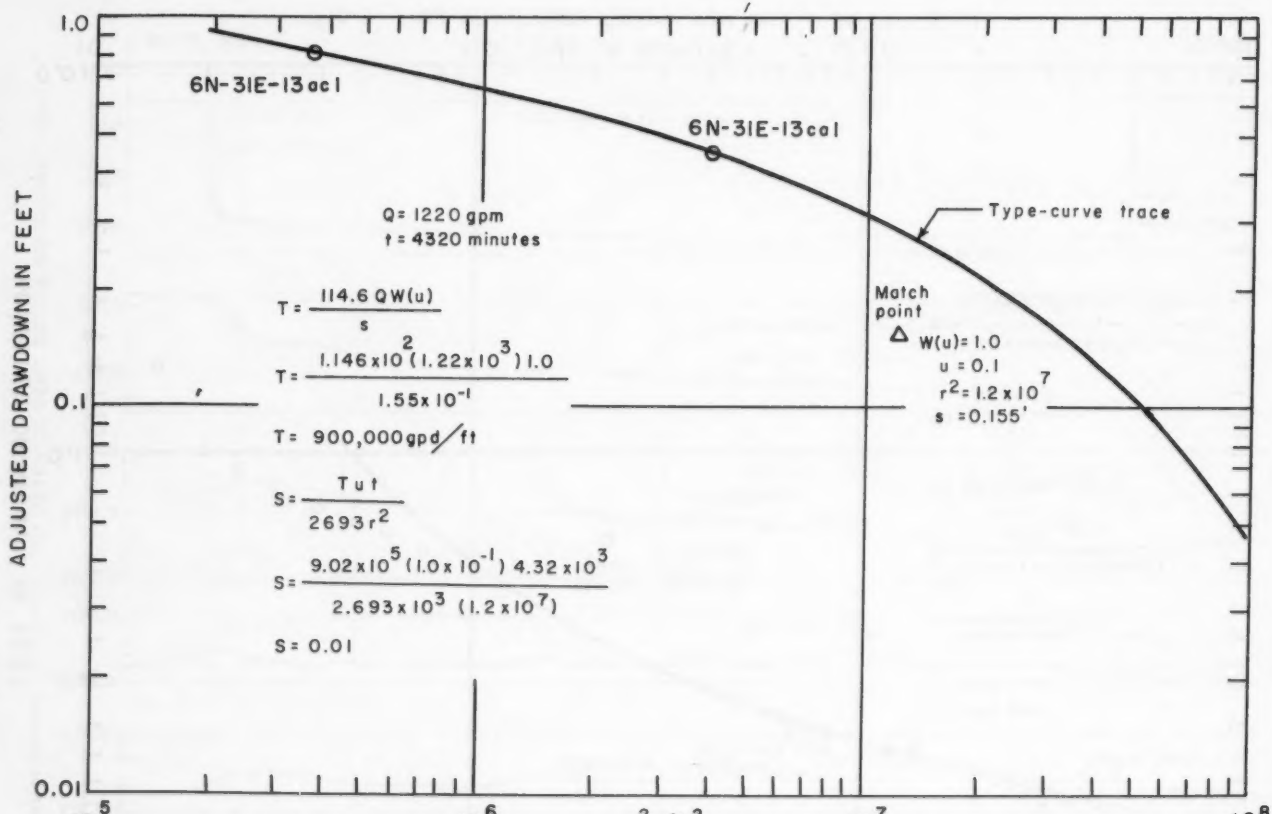

Fig. 5- DISTANCE-DRAWDOWN GRAPH FOR TEST 1.

Table 1 - Coefficients of Transmissibility and Storage for Test 1

Well No.	Type of data	Coefficient of transmissibility (gpd/ft)	Coefficient of Storage
6N-31E-13ac2	Time-drawdown	800,000	--
6N-31E-13ac1	Time-drawdown	800,000	0.02
6N-31E-13ca1	Time-drawdown	700,000	.02
6N-31E-13ac1 and 6N-31E-13ca1	Distance-drawdown	900,000	.01
	Average (rounded)	800,000	0.02

as close as can be expected. However, as is obvious from the preceding discussion, the performance of a well at any particular site cannot be predicted accurately by using the computed values of T and S because of local irregularities in the hydraulic properties of the basalt.

The time-drawdown data did not indicate changes in the water-bearing properties of the basalt great enough to approximate the effect of an impermeable boundary. A boundary of this nature would distort and cone of influence, and the distance-drawdown data would yield values of T and S not in agreement with the values of T and S computed from unaffected time-drawdown data. The fact that the values of T computed by using both time-drawdown and distance-drawdown data agree lends support to this interpretation.

It is unfortunate that at least three wells at different distances from the pumped well were not available for the test, because distance-drawdown curves based on at least three points are desirable. It is seldom, however, that more than one observation well is available for measurement during a test on the Snake River Plain.

Cones of influence do encounter boundaries during aquifer tests in basalt in some places. The effects on water levels of the presence of boundaries are illustrated in the results of an aquifer test made in the southern part of the NRTS area.

An aquifer test (test 2) was made on June 7-11, 1957, with the wells shown in Figure 6. The effects of the discharge of well 3N-29E-14ad2 were measured in wells 3N-29E-14ac1 and 3N-29E-14ad1. Generalized graphical logs of the wells are given in Figure 7. Pumping was started at 10:50 a.m. on June 7 and was continued at a constant rate of 4,350 gpm for a period of 48 hours. Pumping was stopped at 10:50 a.m. on June 9 and the water level in the aquifer was allowed to recover. Figure 8 shows the effects on water levels caused by operation of the pump.

Drawdowns in the observation wells adjusted for atmospheric-pressure changes were plotted against time on logarithmic paper. As an example, the time-drawdown data for well 3N-29E-14ad1 are shown in Figure 9.

After pumping started, the water level in the observation well declined for a time under the influence of the pumped well only. After about 100 minutes of pumping, the rate of drawdown increased under the influence of an

Fig. 5- DISTANCE-DRAWDOWN GRAPH FOR TEST 1.

Fig. 6. Map Showing Locations of Wells Used in Test 2

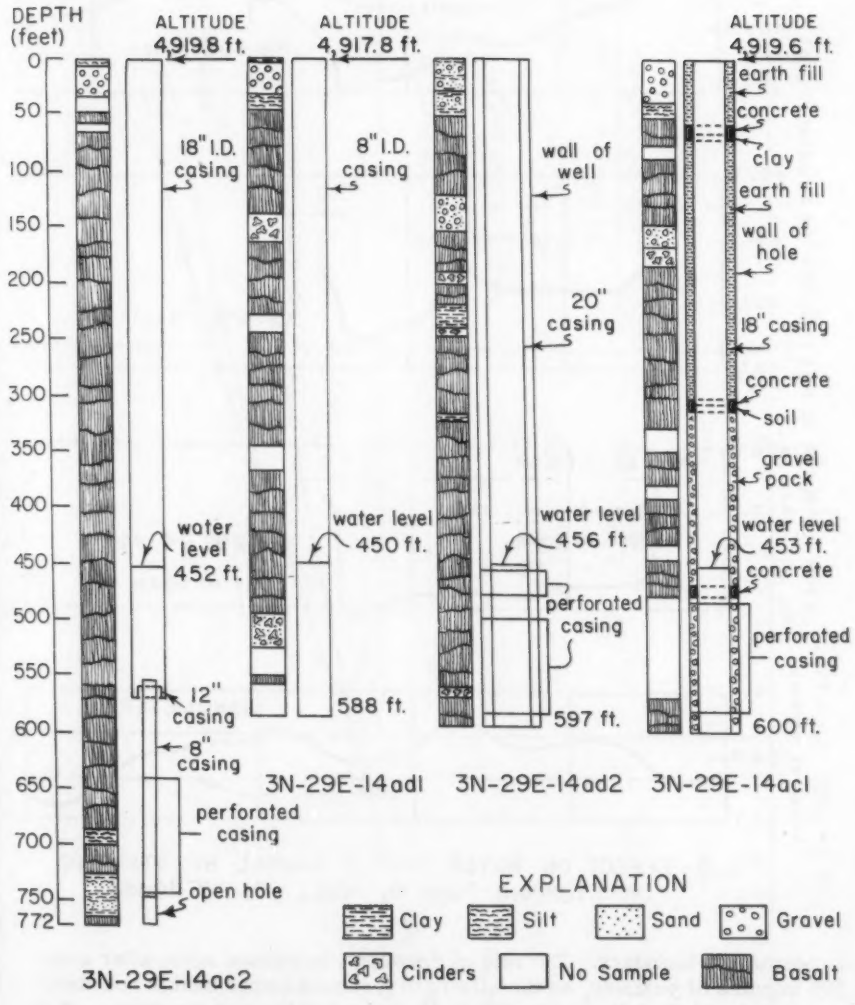

Fig. 7 - GENERALIZED GRAPHIC LOGS OF WELLS USED IN TEST 2.

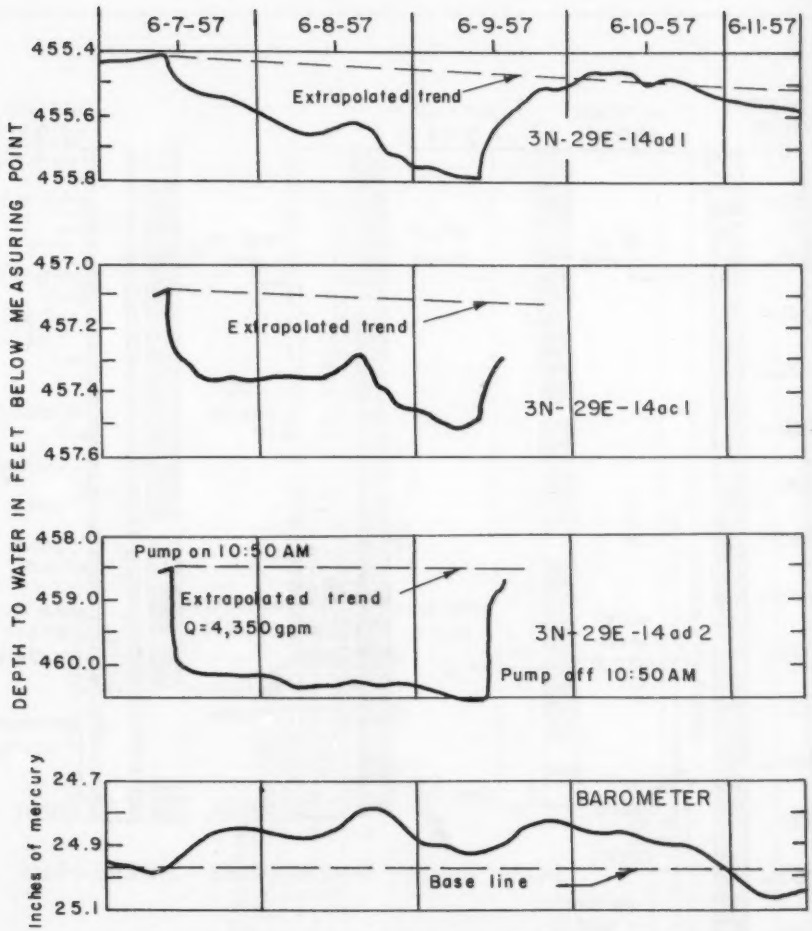

Fig. 8- EFFECT ON WATER LEVELS CAUSED BY STARTING OR STOPPING PUMP IN WELL 3N-29E-14ad2.

impermeable boundary. The rate of drawdown increased again after about 300 minutes of pumping, as the effects of a second impermeable boundary reached the observation well. Thus, by the end of the test the decline of the water level in well 3N-29E-14ad1 was influenced by the pumped well and by two impermeable boundaries. The time-drawdown data indicate that changes in the water-bearing properties of the basalt great enough to approximate the effect of impermeable boundaries occur in two different directions and at relatively short distances from the pumped well.

The nonequilibrium type curve was matched to the portion of the time-drawdown data unaffected by the boundaries and match-point coordinates were used to compute the coefficients of transmissibility and storage of the

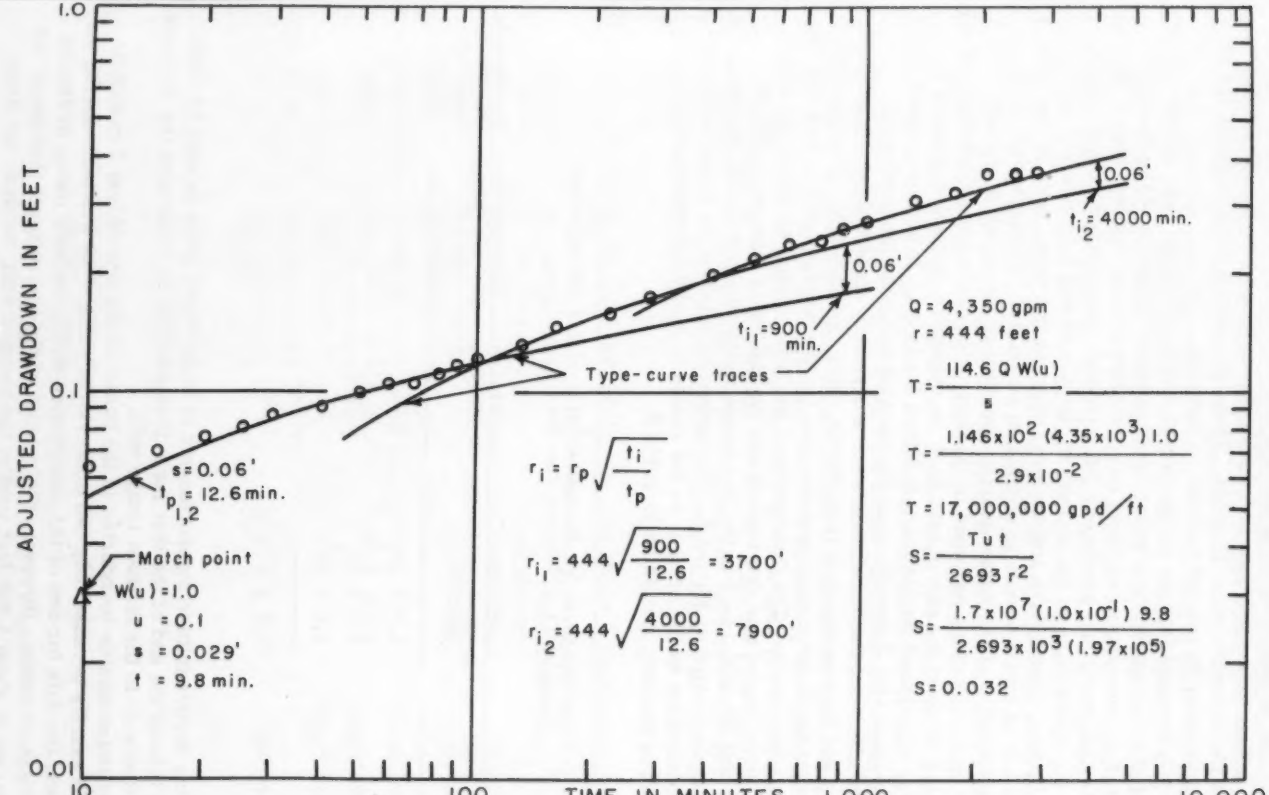

Fig. 9- TIME-DRAWDOWN GRAPH FOR WELL 3N-29E-14adl

aquifer. The time-drawdown data for the first 20 minutes of pumping are interpreted as departing from the type curve because of the differences between actual field conditions and the ideal conditions assumed in the derivation of the nonequilibrium formula. Thus, the curves were matched for the period between 20 and 100 minutes of pumping.

It will be noted that the initial period during which the cone of depression adjusted to the nonuniform water-bearing characteristics of the basalt (20 minutes) is interpreted as being much shorter during this test than it was during the test described previously (90 minutes). According to the driller's logs, the water table in the vicinity of the site of test 2 occurs in a zone of soft, broken, porous lava and cinders, whereas the water table in the vicinity of the site of test 1 occurs in slightly to moderately jointed basalt. The characteristics of the water-bearing openings of the basalt at the site of test 2 are more like those assumed in the derivation of the nonequilibrium formula than are those at the site of test 1. It is, therefore, logical to assume a shorter adjustment period for test 2 than for test 1. In addition, the observation wells of test 2 are closer to the pumped well than those of test 1.

The effects of the boundaries were analyzed by means of the image-well theory and the law of times (see Ferris, 1951, pp. 247-259). The type curve was matched to the time-drawdown data for the portions of this test affected by the image wells. The divergence of the time-drawdown curve from the type-curve traces was determined and the distances from wells 3N-29E-14ad1 and 3N-29E-14ac1 to the two image wells was calculated, by use of the method illustrated in Figure 9. The distances between the image wells and the observation wells, as well as the values of T and S determined from the results of the test, are listed in Table 2.

Table 2 - Coefficients of Transmissibility and Storage and Image-Well Distances for Test 2

Well No.	Coefficient of transmissibility (gpd/ft)	Coefficient of Storage	Distance to closest image well (feet)	Distance to farthest image well (feet)
3N-29E-14ac1	1.7×10^7	0.08	3,000	--
3N-29E-14ad1	1.7×10^7	.03	3,700	7,900
3N-29E-14ad2	2.0×10^7	--	--	--
Average	1.8×10^7	0.06		

Possible errors in tape measurements of water level made in well 3N-29E-14ac1 toward the end of the test made it impossible to determine the distance from that well to the farthest image well.

The impermeable boundaries in the basalt at the site of test 2 cannot be located exactly because only two observation wells were available during the test, and the data for one of the observation wells are poor owing to erratic tape measurements. However, the results of the test suggest that one of the boundaries is about 1,600 feet west of the pumped well, because two arcs

associated with the closest image well nearly intersect west of and about 3,200 feet from the pumped well. The data indicate also that the distance from the pumped well to the other boundary is roughly 4,000 feet. The orientations of the two boundaries cannot be determined.

The poor performance of production well 3N-29E-14ac2, about 830 feet west of well 3N-29E-14ad2, supports the conclusion that a boundary does exist west of the pumped well. Well 3N-29E-14ac2 was originally drilled to 601 feet, about 145 feet below the water table, and had a specific capacity of 12 gpm per foot of drawdown for a pumping period of 24 hours. The well was deepened to 772 feet and tested again. The specific capacity was not increased. For comparison, the specific capacity of well 3N-29E-14ad2 for a pumping period of 24 hours was 2,175 gpm per foot.

Obviously the idealized boundaries determined from the results of aquifer tests rarely if ever describe the actual geologic boundaries of the aquifer, but they represent a hypothetical aquifer system that is equivalent hydraulically to the real system. This hypothetical hydraulic system can be used in estimating at least the approximate magnitude of the effects of long-term pumping from the real system.

A summary of the aquifer-test data collected to date on the Snake River Plain and the coefficients of transmissibility and storage of the Snake River basalt computed therefrom are given in Table 3. Few data are available for areas on the Snake River Plain outside the NRTS. It should be emphasized that the values listed in the table are provisional and subject to revision as further field data are obtained and more is learned about the occurrence and movement of water in the basalt.

Table 3 - Results of Aquifer Tests in the Snake River Basalt

Location of test	Wells used in test	Coefficient of transmissibility (gpd/ft)	Coefficient of Storage
Minidoka	8S-24E-8ad1 8S-24E-8ad2	4.8×10^6	0.04
Mud Lake basin	5 wells in Owaley Canal Co. well field north of North Lake in Jefferson Co.	1.1×10^7	.06
NRTS area	6N-31E-13ac2 6N-31E-13ac1 6N-31E-13ca1	8.0×10^5	.02
	3N-29E-14ac1 3N-29E-14ad1 3N-29E-14ad2	1.8×10^7	.06
	3N-29E-24ad1 3N-30E-19bc1 3N-30E-19cb1	3.3×10^6	.06
	5N-31E-10cd1	5.7×10^5	---

NRTS area	5N-35E-13dbl	7.5×10^5	-
	4N-30E- 7adl	1.7×10^6	-
	4N-30E-30aal	1.5×10^6	-
	4N-30E-30adl	3.7×10^6	-
	2N-29E- 1dbl	1.6×10^5	-

Values of the coefficient of transmissibility of the basalt aquifer given in the table range between 1.6×10^5 and 1.8×10^7 gpd per foot. The average value of T is 4.2×10^6 gpd per foot. For the wells tested the average penetration below the regional water table is about 130 feet. The aquifer ranks as one of the most productive in the United States.

For many of the tests made at the NRTS no observation wells were available and measurements of drawdown were obtained only in the pumped wells. The coefficient of storage of an aquifer cannot be determined by use of data collected in a pumped well unless the effective radius of the well and the entrance losses of head are known. Satisfactory approximations of these quantities can be obtained only under favorable conditions.

The coefficients of storage computed from test data are all indicative of water-table conditions, as they range from 0.02 to 0.06. The coefficient of storage of the Snake River basalt is comparatively low for a water-table aquifer, though reasonably high for one in consolidated rock. Storage coefficients exceeding 0.20 are common for unconfined sand and gravel aquifers.

The coefficients of storage computed from the test data pertain in large part to the portion of the aquifer through which the water table moved during the tests and may not be representative of the basalt as a whole. Further, because of slow draining, longer pumping tests might yield coefficients of storage even larger than those computed.

Factors Complicating Analysis of Aquifer-Test Data

The drawdowns in observation wells during aquifer tests in the Snake River basalt are small, generally less than 0.5 foot, for practical pumping rates. Even though a steel tape is used, unavoidable small errors in water-level measurements are often made because the water table lies rather far below the land surface (the depth to water was about 215 feet in one of the tests described in detail, and 450 feet in the other). Also, precise records of water-level fluctuations are difficult to obtain with float-actuated recording gages because proper balance of floats and counterweights is hard to attain at the depths involved. As a result, floats tend to hang up on the walls of wells and produce erratic water-level records. These unavoidable small errors in water-level measurements amount to an appreciable percentage of the total drawdown, especially during the latter part of tests when the rate of drawdown is small. The data are, therefore, commonly scattered.

The water levels in wells in the Snake River basalt are affected to a great degree by atmospheric-pressure changes. At times, small drawdowns in a well are completely masked by water-level fluctuations caused by atmospheric pressure changes. Consequently, adjustments of drawdown data for

atmospheric-pressure changes are often equal to or greater than the corrected drawdowns themselves.

Adjustments of drawdown data for atmospheric-pressure changes are based on a constant barometric efficiency (the ratio of the change in the water level in a well to the change in atmospheric pressure, expressed in feet of water). For diurnal atmospheric-pressure fluctuations in the summer there is essentially very little difference in the barometric efficiency of a well during periods of increasing and decreasing atmospheric pressure. However, during the winter when pressure changes are large and often erratic, the barometric efficiency of a well is not the same at all times. As a result, water-level data, adjusted for changes in atmospheric pressure based on a constant value of barometric efficiency, are often irregular, which further complicates the analysis of the data.

Specific-Capacity Data

Specific-capacity data obtained during well-acceptance tests can be used to augment aquifer-test data in determining the approximate range of the coefficient of transmissibility (Theis and others, 1954). The specific capacity of a well may be defined as the yield of the well per unit of drawdown and is generally expressed in gallons per minute per foot. A high specific capacity indicates that the aquifer supplying the well has a high coefficient of transmissibility.

Factors other than the coefficient of transmissibility, however, bear upon the specific capacity of a well. The coefficient of storage, the depth of penetration of the well below the regional water table, the size of the well, the entrance losses, and the duration of the pumping period also affect the specific capacity. However, the approximate relation between the coefficient of transmissibility and the specific capacity of a well can be determined from the results of aquifer tests. This relationship can then be used to obtain a rough estimate of the coefficient of transmissibility of the basalt in the vicinity of a well in an area where aquifer-test data are not available but specific-capacity data are.

Table 4 summarizes data obtained to date on the specific capacities of production wells in the Snake River basalt.

The tabulation indicates that, for the Snake River Plain as a whole, the average yield of the basalt to a 16-inch well during a well-acceptance test of about 8 hours is about 2,100 gpm per foot of drawdown. The average depth of wells below the land surface and average penetration below the regional water table are 290 feet and 100 feet, respectively. Although specific capacities ranged from 6 to 22,000 gpm per foot, the common range in specific capacity was from 60 to 3,200 gpm per foot.

Judging from the common range and average of the specific capacities of wells, the coefficient of transmissibility of the Snake River basalt ranges between 1×10^5 and 7×10^6 and averages about 4×10^6 gpd per foot. The wells for which specific capacities and aquifer-test data are available penetrate the basalt only partially. Massive layers of relatively impermeable basalt commonly isolate the lower part of the aquifer from the material penetrated by wells. It is probable, therefore, that the coefficient of transmissibility of the entire thickness of the Snake River basalt greatly exceeds the value determined from data on partially penetrating wells.

Table 4 - Specific-Capacity Data for Wells in the Snake River Basalt

Location of wells	No. of wells for which data are available	Average depth of well (feet)	Average penetration of well below water table (feet)	Average diameter of casing (inches)	Average length of test (hours)	Range of specific capacity (gpm/ft)	Average specific capacity (gpm/ft)	Average pumping rate during test (gpm)
1. Jefferson County	5	200	72	16	16	33-14,000	3,200	2,400
2. U. S. Bureau of Reclamation Minidoka Project, North Side Pumping Division.								
Group 1	15	266	83	21	--	110- 9,000	2,300)	2,900
2	21	264	94	24	2 1/2	110-20,000	4,500)	2,500
Minidoka, 3	29	282	91	21	3	200- 6,000	1,700)	2,200
Jerome, and 4	31	276	88	21	3	59-14,000	1,800) 2400	1,800
Lincoln 5	22	308	83	21	4 1/2	23-17,000	1,800)	1,700
Counties 6	18	402	112	21	2 1/2	63- 3,000	800)	1,500
7	30	271	71	21	3 1/2	150-22,000	3,900)	1,800
3. Bingham County	15	252	71	18	8	16- 7,700	1,800	1,900
4. NRTS Butte County	17	590	200	16	24	12- 4,000	1,100	1,300
5. Gooding County	9	120	60	12	2	300- 2,700	900	1,000
6. Jerome County	6	362	77	16	3	6- 1,600	900	1,700
7. Mud Lake basin (shallow aquifer) Jefferson County	9	78	57	24	2	260- 2,700	800	4,100
8. Bonneville County	6	207	88	18	4	33- 1,000	500	1,800
9. Blaine County	2	226	179	16	2	37- 110	70	1,400
10. Camas County	3	219	77	16	6	38- 110	60	1,200

The yield of wells in basalt sometimes greatly increase with small increases in the depth of penetration of the well below the regional water table as highly permeable zones are encountered by a well. As an example, well 4N-30E-30aa1 on NRTS was drilled to a depth of 483 feet and showed a specific capacity at that depth of 16 gpm per foot. Later the well was deepened to 535 feet and its specific capacity increased to 2,800 gpm per foot. The yield of a well does not always increase with depth. For example, the yield of a well 3N-29E-14ac2 on the NRTS was not appreciably increased by deepening the well from 601 to 772 feet.

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The field and office work of many members of the U. S. Geological Survey in addition to the authors is embodied in the results of this paper. Several participated in the writing of earlier special reports which have been used as reference material. Grateful acknowledgment is made, therefore, to the following members of the U. S. Geological Survey: J. T. Barraclough, E. G. Crosthwaite, Morris Deutsch, F. E. Fennerty, J. R. Jones, I. S. McQueen, R. L. Nace, R. C. Scott, Rex O. Smith, C. V. Theis, W. I. Travis, P. T. Voegeli, and E. H. Walker. The authors are also indebted for background material used in this paper to Lynn Crandall, Henry Gannett, G. R. Mansfield, A. C. Peale, I. C. Russell, H. T. Stearns, W. G. Steward, and Orestes St. John. Acknowledgment is made to officials of the Atomic Energy Commission, U. S. Bureau of Reclamation, and State of Idaho Department of Reclamation, who were most cooperative and helpful in making data available on well-acceptance tests and in providing the facilities required to make aquifer tests.

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CONSOLIDATION OF IRRIGATION COMPANIES AND SYSTEMS

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ABSTRACT

The problems of the old irrigation systems diverting water from a common source are pointed out, the advantages gained by consolidation of these systems are mentioned and recommendations are made concerning consolidation of irrigation companies and systems. The paper presents some of the information developed by the Committee on Consolidation and Progressive Betterment of Old Irrigation Systems.

INTRODUCTION

Except for the air we breathe, nothing is so important to life and civilization as water--water for drinking, water for industry, water for all of the needs of man. Here in the West, lack of water is now restricting many activities. Agriculture founded upon irrigation is limited to the supply available. Industry must be located where water can be obtained. Cities are transporting water hundreds of miles to meet their domestic and municipal requirements. Competition between agriculture, industry, and other users of water is becoming keen and reuse of unconsumed water is increasing. Agriculture under irrigation now consumes more water than all other uses combined and it will probably continue to be the largest single consumptive user of water. Along with the extensive use of water, irrigation probably is a major source of waste of the valuable water resource due in large measure to the inefficiency of existing canals and distribution systems with their duplication and obsolescence. The problems of these old canal systems must receive more public attention and emphasis.

For many years engineers have been concerned with the problems of small irrigation systems diverting water from a common source, serving the same

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2157 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. IR 3, September, 1959.

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general area and having common problems of operation and maintenance. It was nearly 30 years ago that Dr. O. W. Israelsen* first proposed consolidation to four irrigation companies and four drainage districts in the West Millard county area of Utah. A consolidation that has not yet been achieved even though the institutions involved have all the elements that would appear to dictate consolidation, i.e. common water supply, adjacent lands, common stockholders, duplicated canals and common problems of operation, maintenance and management.

Almost 20 years ago two small irrigation companies in southern Utah were approached by the author concerning their common problems. Both of these companies diverted water from the same small mountain stream. One had the direct flow rights, the other had a small storage reservoir. Over 60% of the stockholders had stock in both companies. To an outsider, it appeared that each company had what the other needed to round out its water supply and to make the irrigation system for the valley more secure. However, for many years these institutions had operated as two separate companies, the reservoir company storing water in the winter and releasing it in early spring for the irrigation of the lands under that company and the direct flow company diverting all of the high spring flows for the irrigation of their lands. By July, after the flood flows had passed, the direct flow company was reduced to its low flow stage and the reservoir company was out of water. By combination and consolidation, a water supply ample for the entire area throughout the irrigation season, could be provided. To the author's knowledge, consolidation has not yet been achieved.

These examples along with countless other examples that could be mentioned makes one ask the question, "Why have these obvious advantages of consolidation been bypassed for so many years?"

In order to answer this question, it is necessary to study the development pattern of our western water supplies. This development has to do not only with the physical development of canals, reservoirs, and other structures, but the legal, human, and economic developments as well.

Physical Development

Beginning about 1850, the pioneer settlers located their diversions and canals low in the valleys adjacent to the lands easily irrigated and built their works accordingly. Later settlers were forced to make their diversions higher on the stream where construction of diversion works and main canals was more difficult and they were forced to select lands more distant from the stream requiring larger and longer canal systems. These systems were largely cooperative enterprises planned to solve the problems of the day for the individual or group. The ultimate development of the water supply of the area was not considered. The result was inevitable, large areas of land were brought under irrigation without the benefit of our present day knowledge, and in many cases without the benefit of the engineering services available at the time. Later developments were seldom integrated into the existing systems. Even though they were planned and constructed many years ago, they have maintained their identity down to the present time. Many systems thus became established on a common water source with diversion dams for each system, parallel canals and duplicating works.

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Many examples of parallel canals and duplication of structures can be pointed out. Figure 1 shows one of these systems. The system shown was originally constructed as a single canal, but within a few years it was reconstructed to form a monument to idiocy that has endured for almost 100 years. These canals have a common bank for almost 5 miles. The heavy growth of phreatophytes on the center bank is almost impossible to control. Enough muskrat holes probably exist between the canals to eliminate the initial purpose of dividing the water between the two groups of water users.

This may be an extreme example of the evils of parallel systems, but it shows plainly the increased costs and inefficiencies that go along with duplicate works. The following problems will be found in areas where parallel and duplicate systems exist whether the canals are side by side or separated.

1. The costs of bridges and culverts is nearly doubled. Each road and highway crossing requires two bridges instead of one.
2. The amount of right of way required is increased and otherwise productive land must be dedicated to canal use.
3. Costly and inconvenient water diversions cannot be avoided. All of the deliveries are to lands below the canals and water diverted from the upper canal must bypass the adjoining system requiring costly and complicated structures.
4. Seepage losses are increased over the losses of a single canal.
5. Operation costs are greatly increased because of inconvenience and duplication.
6. Maintenance costs are also increased because of inconveniences and the impossibility of utilizing to full advantage and the modern mechanical equipment now available.

Legal Developments

Coinciding with the physical development of the water resources, was the legal development of the right to use water. Western states recognize the doctrine of appropriation, which simply stated is "First in time is first in right." The early pioneers who first developed the water obtained the first right to use of the stream, while later settlers acquired junior rights. Many of the original water rights are for direct flow only, while some of the later rights may combine storage rights with flood flow diversions.

The important point in the water right picture is that the mere possession of a water right may not guarantee any water to the owner of the right. When the water supply of a stream fails to satisfy the diversion requirements of existing water rights, the stream, of course, is over-appropriated and junior rights must give way to prior rights. This condition may happen only in dry years on some streams, whereas it may happen every year on others. A good example of this situation can be obtained by superimposing the water right demand on the hydrograph of flow for most any of the western streams. Charts similar to the ones for the Uinta River in eastern Utah will probably be the result. Figure 2 is a daily flow hydrograph of the Uinta River for the year 1935 upon which has been superimposed the water rights of the stream. Even though 1935 is a near normal water year, it will be noted that the water rights exceed the supply except for a short period during June when the flood

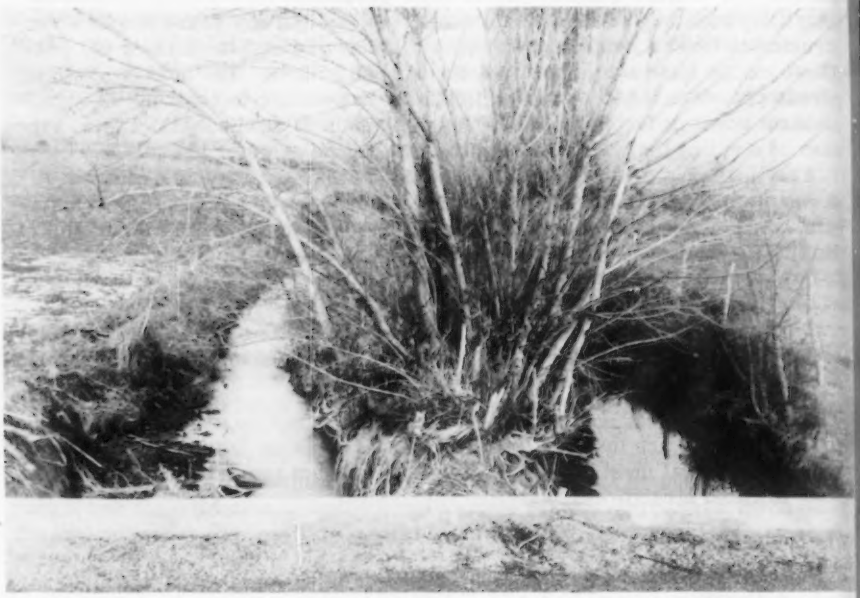


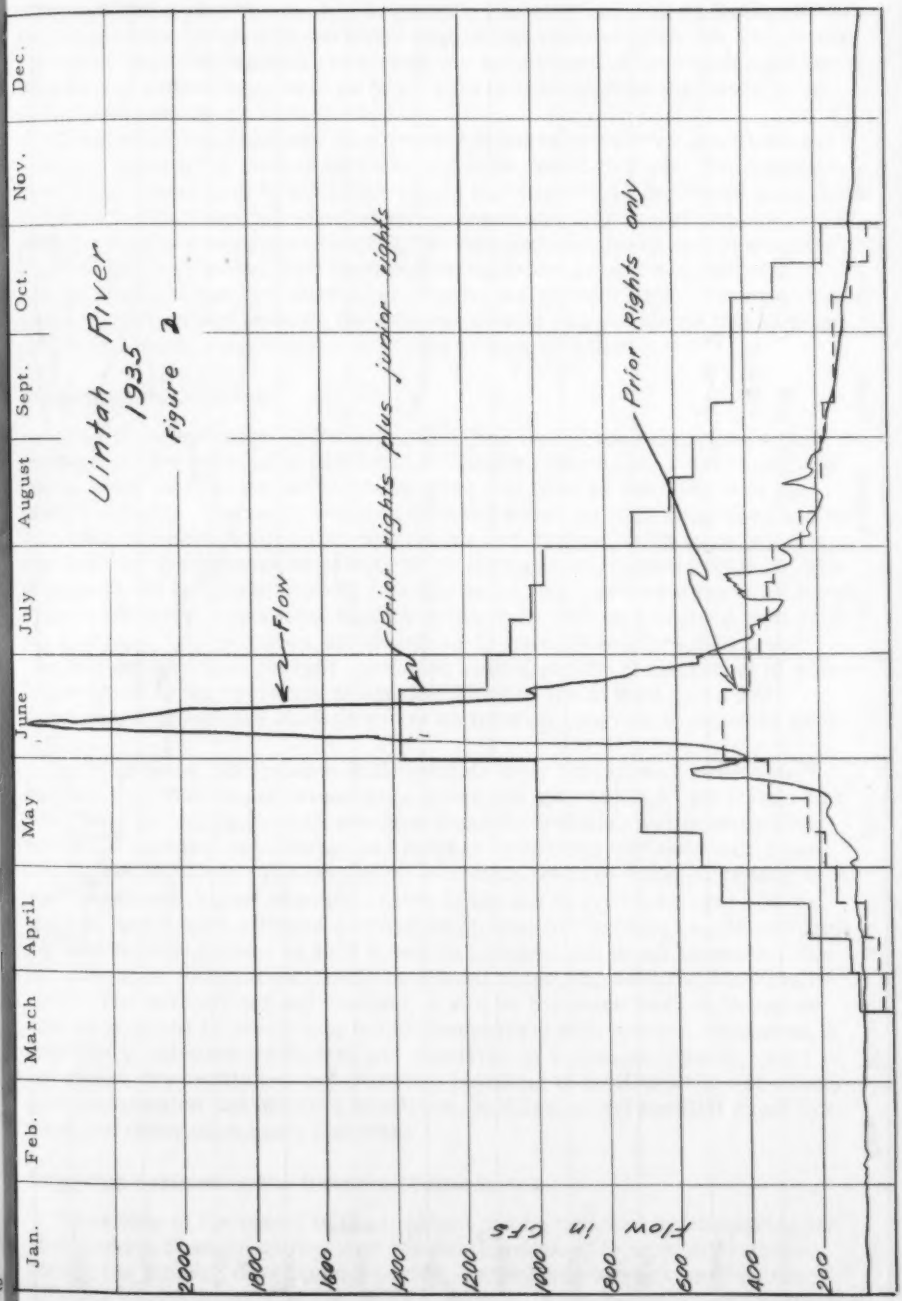
Figure 1

flow caused by melting snow is at its peak. Prior rights during this year suffered only minor shortages before the snow melt season but junior rights were able to divert water only during the flood flow period. Figure 3 is a similar chart for the same stream in a low water year when the supply was not sufficient to satisfy even the prior rights. A court decree in this area limiting the annual diversions to 3 acre feet per acre produced the variation in the seasonal water right picture. Ordinarily the water rights may be represented by straight horizontal line on the chart for the period of use.

Consolidations of irrigation companies will most certainly bring together water rights of different priorities. The evaluation of one right in terms of another will be most difficult, for in a good water year a junior right may obtain as much water as a prior right, but in drought years, the junior right may get no water at all. Any water right evaluation must have as its foundation a complete analysis of the water supply. After consolidation, the water rights must maintain their identity to satisfy legal requirements. Many companies, however, obtain their water supply under several separate water rights and this feature of maintaining the identity of the right creates no problem.

Development of Human Problems

A complicating factor in the combining of irrigation systems is the attitude of the present owner. He has a special relationship to water rights; he


Uintah River
1935
Figure 2
Flow
Prior rights plus junior rights
Prior Rights only
Flow in CFS
2000
1800
1600
1400
1200
1000
800
600
400
200
Jan.
Feb.
March
April
May
June
July
August
Sept.
Oct.
Nov.
Dec.

Uintah River
1934
Figure 3
Jan.
Feb.
March
April
May
June
July
August
Sept.
Oct.
Nov.
Dec.
2000
1800
1600
1400
1200
1000
800
600
400
200
Flow in C.F.S.
Prior rights plus junior rights
Prior rights only
Flow

developed the right. He has had to guard it jealously for fear of losing it. He has adapted his farming to the water supply represented by it. He will probably resist any combinations because of the uncertainty of the result. He knows what he can expect from what he has. This is true whether the owner is an irrigation company or an individual.

Changes and combinations may require changes in farming practices and it is only natural for owners of water rights to resist change. The development of the water supply and water rights has sometimes developed jealousies and hard feelings against adjacent water users and even though the original settlers may in many cases be dead, the antagonisms, fears, and jealousies of the original pioneers have been passed on to their heirs and successors, and the problem has remained alive down to the present time. Thus a human attitude or viewpoint towards the existing system has developed that is deep seated and it will increase the problems of consolidation.

Economic Development

A fourth development is the economic condition of the existing systems. In considering the economics of the old irrigation system, one must recognize the fact that most of the development work was done by the owners of the lands benefited. The early settlers diverted water directly from the streams by means of individually constructed dams and ditches which were planned and built for the purpose of solving their individual irrigation problem. The irrigation works constructed by individuals or small groups were considered private property, subsequent developments were seldom combined with existing systems. The resulting developments in many cases, are debt free. Original construction charges have been repaid and the present cost of water to the users is for operation and maintenance of the system. In a few instances, a group may have an active betterment program to improve their system.

Many of these old systems still maintain their initial usefulness and efficiency, but even so, they may have grown old and outdated. No matter how wise their initial conception, nor how sound their design and construction, irrigation systems deteriorate and modern technology and economic conditions overtake them. The standards of usefulness have changed greatly in the past 50 years. A good example of this is the horse drawn scraper. In its day this was a very efficient piece of equipment for building canals and grading land for irrigation. In 1920 it was in common use by all farmers. The same scraper today, even though in perfect condition, would be very inefficient. The scraper has not changed, it will do the same work at the same rate as it would 40 years ago, but by comparison with modern equipment, it would be considered inefficient and obsolete. In a similar manner, many of the canals are inefficient and obsolete, resulting in inefficient use of water, poor distribution and delivery practices, duplicating and parallel canal systems and other inadequate features.

Irrigation Systems in the Western United States

Some idea of the extent of the problem can be obtained by comparing the development of communities with the development of irrigation systems. During the pioneer development period, settlements were formed on the streams where water supplies were available. Even the smallest creeks have a small community at their mouths and much of the water for irrigation in the

west comes from small mountain streams. Utah now has nearly 400 cities, towns, and villages as the result of this development. By contrast, however, the 1950 census lists 3,165 irrigation enterprises in Utah, 1058 of which are group enterprises. Of these group enterprises in Utah, 406 are unincorporated mutual companies, 634 are incorporated mutual companies, 5 are irrigation districts, 2 are commercial companies, 7 are cities, 3 are irrigation developments of the U. S. Bureau of Indian Affairs and one is a Bureau of Reclamation operated project (Other U. S. Reclamation projects in Utah are operated by Water User's Associations). The area irrigated by the 1058 group enterprises in Utah is 1,042,497 acres making the average 985 acres per enterprise. The 2107 remaining projects are listed as single farm enterprises consisting largely of wells and pumping plants developed for individual farms.

For the 17 western states the 1950 census lists 111,940 separate irrigation enterprises 101,770 of which are single farm enterprises and 10,170 group enterprises. Of these group enterprises 6,417 are unincorporated mutual companies, 2880 are incorporated mutual companies, 131 are commercial companies, 483 are irrigation districts, 37 are U. S. Bureau of Reclamation projects, 141 are U. S. Bureau of Indian Affairs enterprises, 31 are state enterprises and 50 are city operated.

The 10,170 group enterprises in the 17 western states irrigate a total area of 14,713,888 acres or an average area of 1447 acres per enterprise.

Not all of these enterprises, of course, could be considered badly in need of improvement and only a small percentage might be classed as duplications where consolidations would be recommended. It should be pointed out, however, that if as many as 10 percent of the group enterprises fell into this class it would mean that more than 1,000 systems should be consolidated and modernized.

In 1946 Israelsen and others published(1) the results of a survey of irrigation companies in Utah. Data were obtained from 688 separate companies. There is no legal limit to the minimum size of an irrigation company as seen by the fact that 179 of the 688 companies serve areas of less than 300 acres, some even less than 100 acres.

The other 509 serve areas larger than 300 acres, the largest one of which serves approximately 50,000 acres.

To staff the 688 separate companies in Utah requires the services of 2,606 officials. Although water delivery and distribution is considered to be largely an engineering problem only 69 of the 688 companies regularly employed an engineer. The importance of water rights is evidenced by the fact that 167 of the 688 regularly employed attorneys.

The large number of irrigation enterprises and the small area served by group enterprises indicates there is a problem of considerable magnitude concerned with consolidation of irrigation companies and systems.

Consolidation Presents a Challenge

The physical development of the canal systems and appurtenent works, the legal development of the right to use water combined with the human and economic problems have created the present predicament with regards to the older irrigation systems. They are really in a dilemma. In the interest of obtaining maximum benefits from the valuable water resource, something must be done to consolidate, revamp, and modernize these old systems.

However, the reorganization and betterments will certainly call for an initial outlay of capital. To solve these problems, a major challenge is presented to the engineers, lawyers, economists, and water users interested in water resources. How can this challenge be met and what is necessary to get the job done? The author recommends 4 basic steps or activities as follows:

First: There must be a recognition of the problems of the old irrigation systems by public and private groups dealing with the development, use and administration of the water resource. This includes the irrigation company officials and irrigators, Federal and State Agencies, consulting engineers, and lawyers.

Second: There must be an education process to create a desire to consolidate and improve the existing irrigation companies and systems.

Third: There must be a sound consolidation plan for systems to be integrated. This plan must be sound considered from an engineering standpoint, legal standpoint, and economic standpoint.

Fourth: There must be provided a method by which necessary changes can be financed.

In order to adequately present the facts to the water users involved, a careful appraisal must be made of each individual system. The existing physical facilities, water rights, water supply, legal problems and special problems must be inventoried. Since the water supply is the foundation of the enterprise, an evaluation of the water supply with respect to its occurrence in time and amount is necessary in order to evaluate the comparative desirabilities of the systems to be integrated. Hydrologic studies must be made where data are not available and where water supply information is available, it must be carefully studied and evaluated.

Potential water supplies must be investigated such as storage, groundwater development, and transdiversions. Methods must be found for improving the efficiency of existing water conveyance and use through canal lining or changes in administrative procedures. Methods of delivery in use by the companies must be studied in relation to the water supply to determine if changes might be made that would be advantageous. In short, all of the facts connected with the water resource must be obtained.

The physical facilities of the irrigation companies must be appraised to determine their adequacy and value. Where duplications and overlapping systems are evident, designs, and cost estimates for a comprehensive system must be made. Existing structures should be studied to determine their adequacy based on present day standards. Additional devices needed to control and measure the water efficiently should be determined. Surveys to improve canal alignments must be made. The operation costs of the existing facilities along with the maintenance problems must be evaluated. Costly operation or maintenance practices that will be eliminated by consolidations might be discovered. This is an important feature and may be a means whereby the engineers and others can obtain cooperation towards consolidation from the water users involved.

The legal problems will be many. The relation of the water supply to the water right and need and the method of combining the water rights for the integrated companies must be established. The identity of the water right must be maintained, but the water represented by the water right must be

combined and redistributed according to requirements of the water users under the combined system. The pooling of stock having different priorities or different basic values must be worked out and stock re-issued having an equal par value and representing the same quantity of water per share. The rights of way problems involved in new construction will be complicated.

Finally, the economics of consolidation must be clearly outlined. The cost of construction and the benefits must be determined. The savings of water affected by eliminating overlapping systems must be evaluated in terms of the savings anticipated and the cost of the construction required to bring about the savings. Savings in water, savings in operation, and savings in maintenance must all be evaluated in terms of dollars. Costs of new construction, costs of new programs, and changes necessary to modernize the system must be carefully estimated. The financial conditions of each company must be determined and debt obligations liquidated or adjusted within the framework of the consolidation proposed. The economics of the area and the economics of the country must also be given consideration. Considering all economic features, to have a case for consolidation, the benefits must exceed the costs.

These particular studies become more involved when it is realized that each combination or consolidation presents unique problems. The situation cannot be studied in one area and solutions applied to other areas as a blanket rule. In every consolidation scheme, the merits of the consolidation must be considered individually. Thus, every area where duplicating systems exist, must be given the same detailed consideration with regards to engineering facilities, economic factors, legal implications of water rights and rights of way in the benefits to be derived from the consolidation.

After all of the field surveys have been made, the water supplies determined, the water rights evaluated, the economic analysis carefully prepared, the greatest problem still remains. That of harmonizing the conflicting interests of the groups involved, into a unified purpose. This involves the delicate handling of the human problem that has developed and has confounded most of the consolidation attempts in the past. One would think that a preponderance of economic evidence and education of the groups concerned would make this problem easy. However, many of the irrigators will maintain a skepticism in the face of overwhelming economic evidence. This has often led to the following comment, "Before consolidation of these irrigation companies can be achieved, there must be several funerals." In some cases, the funerals have long since occurred, yet the consolidation has not yet been completed. The prejudices and jealousies of the leaders of the companies were passed on to their successors and are still evident, remaining as a major obstacle in the path of any consolidation attempt. In such cases, consolidation must wait until economic pressure or public sentiment forces the irrigation companies to utilize their water supplies more wisely and efficiently.

New Projects Offer Promise for Consolidation

The greatest possibility for consolidation will probably be realized in the construction of new facilities or major replacements of existing structures. Where some major project involving all concerned can be designed, a common ground is established to bring conflicting interests together. This offers an excellent opportunity to unite all groups involved under the new project. Where such projects are now under construction by public agencies or private

groups, the possibility of combining small irrigation companies into larger more efficient organizations should not be bypassed even though consolidation would require considerable time in negotiations, education, and legal work. Storage reservoirs, regulation reservoirs, diversion dams, transdiversions to develop new water, and canal lining projects all offer possibilities in this regard. In two instances in Utah, small companies diverting water from the same stream have been consolidated by such a means. In one case a small storage reservoir was constructed by the irrigation companies and after the construction of the reservoir one enterprise emerged to combine the new water made available through the project with existing supplies and distribute it to the water users. On many of the Bureau of Reclamation projects the same prospect is in evidence if properly handled. The large multiple purpose projects and the large area served by the reclamation works offers an excellent opportunity to unite many small irrigation enterprises together in a district for the distribution of the water under the new project. At the same time, the consolidation of the rights, water supplies, canal systems, etc., of the duplicating enterprises should be developed. The prospects would be better at this time than at any other opportunity afforded. To attempt to consolidate merely by compiling overwhelming evidence concerning the advantages, will not completely do the job. More success will be obtained by combining consolidation with some project to unify the interests of the water users toward a common goal.

Interest Growing in Consolidation Problems

In 1950 the irrigation division of the American Society of Civil Engineers established a technical committee on consolidation and progressive betterment of old irrigation systems. This was perhaps the first organized approach to the problems of consolidation of irrigation companies and systems. The objectives of the committee were defined as follows:

1. To develop basic principles upon which the consolidation of irrigation enterprises using water from a common source and having common problems can be proposed.
2. To develop methods and procedures for carrying out progressive betterment or major improvement programs on old irrigation systems within the financial limitations of the enterprise over a period of years.
3. To consider the problem of rehabilitation of irrigation systems that were built many years ago and without the benefit of engineering services.

The committee has done considerable work but because of the complexity of the problem, progress is slow. Other organizations are now becoming interested in this problem. To add momentum to the consolidation crusade, it may be desirable to recommend that Federal and State agencies develop some inducements to encourage consolidation and betterment of irrigation systems. These inducements could take the following form:

1. Financial subsity by making non interest bearing loans.
2. Providing engineering and legal assistance.
3. Cost sharing as in the case of the Agricultural Stabilization program.

Public recognition and public support to the problems of consolidation of irrigation systems is necessary in addition to the services of engineers and attorneys dealing directly with the problem. Until this can be achieved, progress will continue to be slow.

Advantages of Consolidation

Where consolidation can be achieved, the committee believes the following advantages will result:

1. Increased efficiency in the use of the existing water supply by eliminating duplicate systems and making other improvements.
2. Increased efficiency in the management of irrigation enterprises by reducing the total number of water masters, board members and other officials required.
3. Increased efficiency in the operation of irrigation systems by having projects of sufficient size to justify the employment of technically trained engineers and others.
4. Increased safety in operation by modernizing and improving the physical facilities.
5. Increased flexibility of the water supply to meet the agricultural requirements.
6. Increased strength and effectiveness of the irrigation institution.

Consolidation of irrigation systems offers a fertile field for the more efficient utilization of water resources. In discussing this problem before the Irrigation Division of the American Society of Civil Engineers in Salt Lake City in 1954, George D. Clyde, now Governor of the State of Utah and former Chief of the Division of Irrigation and Water Conservation of the Soil Conservation Service, had this to say, "Consolidation of small irrigation companies offers great opportunities to increase the net usable water supply available for irrigation and presents a challenge to the ingenuity and ability of the engineers, the cooperation and progressiveness of the water users and the ability of the legal profession."

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1. Irrigation Companies in Utah, their Activities and Needs, Utah Agricultural Experiment Station Bulletin 322, by O. W. Israelsen, J. Howard Maughan, and George P. South.

Journal of the
IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

DRAINAGE IN THE COACHELLA VALLEY OF CALIFORNIA[a]

Lowell O. Weeks[1]

INTRODUCTION

Coachella Valley is located in the central portion of Riverside County, within a basin which was called Salton Sink on most maps around the turn of the present century. Roughly, this inland desert valley, some 50 miles long and 3 to 9 miles wide, lies between the Little San Bernardino Mountains on the east and the Peninsular Range on the west. The rugged San Bernardino mountains are to the north and east. The Salton Sea, which lies both in Riverside County and Imperial County forms Coachella Valley's southeastern boundary.

The climate of Coachella Valley is characterized by long, extremely hot summers, with occasional high temperatures throughout the entire year, mild winters, a low relative humidity, cloudless skies and an almost negligible rainfall. Frosts are rare, with killing frosts likely to occur, when they do occur, from the first of December until the middle of February. Every month has had a temperature to reach at least 90 degrees. Owing to this high temperature and almost daily sunshine, evaporation is very high during every month of the year and the need for irrigation is year around due to the long or yearly irrigation season.

The average rainfall is so slight in Coachella Valley that it is practically disregarded and the sole dependence for water in growing crops is placed in irrigation. The sources of this supply are deep irrigation wells and, since, 1949, in a supply from the Colorado River diverted into the All-American Canal at Imperial Dam, the location of which is approximately 20 miles upriver from Yuma, Arizona, and then through the All-American Canal a distance of 150 miles to Coachella Valley where it is distributed onto the agricultural lands through a low-pressure concrete irrigation pipe line system.

At an early date in the development of irrigation in Coachella Valley it was believed the need for drainage would arise. In 1927 a number of shallow, perforated-cased, ground water observation wells, called "Alkali Wells,"

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a. Presented at the February, 1959, Convention in Los Angeles, California.

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were installed, ranging in depth from 11 to 32 feet. The water levels in these wells were recorded for several years but with the diminishing water levels in the irrigation wells, due to an overdraft, interest was lost in the "Alkali Wells" and by 1940 most of them were lost or damaged beyond repair.

As long as all of the farming, approximately 17,000 acres had been dependent for its source of irrigation water from ground-water, drainage was not an important problem, partly because of the effect of heavy pumping on artesian pressures. However, with the approach of Colorado River water with its heavy content of total salts, interest in drainage became very important. This new supply would cause an expansion in farming, change irrigation practices and water tables would rise which would cause serious drainage difficulties in the future. On July 1, 1945, a Memorandum of Agreement was signed by the California Experiment Station, the United States Salinity Laboratory of the Department of Agriculture and the Coachella Valley County Water District relative to "Investigations on Salinity Removal and Control by Drainage." In 1948 the Bureau of Reclamation also joined in this agreement and the group is now known as "Coachella Valley Drainage Cooperators." All of the agencies are concerned with the research and investigational phases of this work but the Water District is solely responsible for the design and construction of drainage works.

In order to achieve the purpose of the Memorandum of Agreement it was considered necessary to have a complete knowledge of the hydrologic and subsurface profile of the valley. This would give information regarding the objectives set out by the Cooperators which are: (a) to observe ground water conditions; (b) to develop techniques for land reclamation of salt-affected soils; and (c) to obtain information for the practical design of drainage systems.

In order to obtain knowledge of the ground water, its elevation and direction of movement, a network of shallow ground water observation wells were installed. The first wells were drilled and cased with a 2-inch diameter asbestos-cement pipe. These were installed on a two mile grid. The next group of observation wells installed were small diameter pipe, piezometers, which were jetted into position (Reger and others, 1950). For ground water observation records, wells are now in place near the center of each section and also at each section corner.

The installing of several piezometers at one location with varying depths makes possible a detection of free and confined water bodies, perched and semi-perched water bodies and variations in pressure head with depth below ground surface. Vertical as well as horizontal direction of flow are noted as well as the elevation of the water table. As the piezometers are jetted into place the qualitative nature of the material penetrated can be detected. A complete log of each piezometer is made as to the material of the subsurface and the depth of each textural change. Such information has proven very valuable in studying the movement of ground water and applying corrective measures to the solution of drainage problems.

Reclamation of Salt-Affected Soils

Soils in Coachella Valley are chiefly formed from materials washed down from the mountains on the west and the northwest sides of the valley. These soils are predominantly sandy and have medium to low moisture retention properties. They are deficient in organic matter; they are highly micaceous

and irregularly stratified with no uniformity in thickness, texture or order of strata arrangement. All have been derived from unconsolidated materials transported and redistributed by wind or water.

However, many of the soils in the lower area of the valley are high in soluble salts and either will be or already have been placed into agriculture as a result of the construction of the distribution system which makes irrigation water available for these lands. Early development within the valley was restricted by the lack of ground water for irrigation, which resulted in the development of the better soils.

The problems relating to the reclamation of salt-affected soil have been studied in some detail by the Cooperators and experiments relating to the removal of excessive salts from the salt-affected soils were carried out (Reeve and others, 1955). The Colorado River water, even though it has approximately one ton of total soluble salts per acre-foot of water, is high in gypsum and is a very good reclamation water. From the data obtained from the many experiments, it is the Cooperators recommendation that the ponding of water on the ground surface, leaching, which dissolves and pushes the salts vertically downward is the method that results in successful land reclamation.

Drainage Control

The most important need for land drainage in Coachella Valley is to remove the soluble salts from the soil. In most soils and natural waters in arid regions, there is contained a large percentage of soluble salts. Because of insufficient precipitation, these salts have not been washed from the soil as is the case in humid sections of the world. Also, when the elevation of the water table nears the surface of the ground, the water is drawn by capillary action to the surface where it is evaporated and a deposit of salt is left upon the soil surface. At the present time the drainage control system that has been installed to control the level of the ground water, and to remove the soluble salts, has been the underground farm tile drainage systems. These have been adequate to control the drainage problem and are an absolute necessity in the reclamation of areas with a high water table and soil conditions similar to those of the Coachella Valley.

The farm tile drainage system is laid out in a grid arrangement with a base line or collector line of 8-inch diameter pipe with laterals of either 4, 5, or 6-inch diameter pipe depending on the length of the lateral. These drain pipes are made of either red clay or type V cement. The average depth of the drain tile is 7 feet below the ground surface and the spacing is approximately 275 feet. The actual spacing depends upon the permeability of the soil but for all practical purposes, because the soil profile is so stratified, practicability and not permeability determines the spacing. These farm drainage systems discharge into the outlet drainage system as provided by the Water District.

District Outlet Drainage System

The main outlet drains for all farm tile drainage systems are constructed and maintained by the Water District. All of the drainage water discharge into Salton Sea which is the drainage basin for both the Imperial and Coachella Valleys. The outlet policy of the District is one outlet for each 80 acres or to within one quarter of a mile of each 40 acre parcel.

The outlet drains are either open drains or underground pipe drains. The open drains are used in the area between the agricultural lands and the Salton Sea and as large valley-wide collector drains where it is not feasible to use pipe. In the rest of the valley underground pipe drains are used as the outlet system. The completed system will be composed of 250 miles of pipe drains and 75 miles of open drains of which 25 miles is the Whitewater River flowing through the center of the Valley.

Design Criteria

Another function of the Coachella Valley Drainage Cooperators was to obtain information for the practical design of drainage systems. Data on which to base the design capacity of tile drains in irrigated areas have been lacking. The "drainage coefficient" used in humid areas is not applicable. Measurements of drainage discharge from many farm tile drainage systems were obtained and these have been plotted and a simple equation has been developed, to give maximum discharge in terms of drainage area or length of tile (Pillsbury and others, 1956). The emprical equation is

$$q = 50L^{-0.25}$$

where q = maximum discharge in gallons per thousand feet of tile, and L = length of tile in thousands of feet. Expressed on an area basis, and using an average of 6500 feet of tile per 40-acre parcel and adjusting to a tabulation for easy use in the District Engineering Section the discharge requirements are shown as follows:

Acreage	*Required Capacity*
0-40 Acres	*0.4 cfs*
41-80 Acres	*0.7 cfs*
81-999 Acres	*0.2 cfs for each additional 40-acre parcel*
1000-3000 Acres	*0.1 cfs for each additional 40-acre parcel*

It is possible that this discharge formula may be somewhat conservative but such is necessary because of uncertainties in predicting future drainage needs. It is anticipated that it will be improved and refined as additional acreage is tiled drained and as additional records are obtained.

The Manning Formula

$$Q = \frac{0.463}{n} d^{8/3} s^{1/2}$$

is used for the design of pipe size for the underground drains. In the formula the required capacity and the slope are known, keeping in mind that the farm tile drainage system must have a 7-foot outfall. The only varible in the formula left in doubt was the Manning friction factor "n." The literature had little to say about the friction factor in open-joint drain pipe. Therefore, it was decided that a field test was required to obtain the correct answer. In

1954 a badly needed drain was constructed upon which the tests could be made. This drain was 9300 feet in length, composed of 16-inch diameter concrete tile pipe layed on a slope of 0.00575 by the conventional tile laying method. The tests were made by the Bureau of Reclamation's Hydraulic Laboratory and the Water District, working together. The results of a pipe flowing full for the Manning "n" was 0.01152 (McBirney, 1954). An additional set of tests on the same drain has just been made by the Bureau of Reclamation and the Water District to check for any change due to pipe age. These results are not available at the present time. Because of the many unknowns in designing of farm drainage and also as an added safety factor, the District uses a Manning "n" of 0.0125.

Drain Pipe Specifications

As the sulfate content of the drainage waters is high it was determined at the very start of the drainage program that a dense pipe made under rigid specifications would be required for all pipe to be used in the outlet system. The absorption test was used for determining denseness but difficulty arose when it was found that samples of pipe that passed the absorption test would not always pass the internal hydrostactic pressure test, but all pipe samples that passed the hydrostactic test would pass the absorption test. Also as the District intended to use pipe up to 24-inch in diameter under difficult load depth it was decided to require an extra thick shell. Therefore, the requirements are that all drain pipe shall conform to the requirements of the "Standard Specifications for Concrete Irrigation Pipe" (ASTM Designation C118-52) of the American Society for Testing Materials with the added provision that the cement used shall be Type V, Table I of the ASTM specifications is deleted and the following table is substituted.

PHYSICAL TEST REQUIREMENTS FOR STANDARD CONCRETE IRRIGATION PIPE

		TEST REQUIREMENTS	
Internal Diameter, in.	*Minimum Shell Thickness, in.*	*Internal Hydrostatic Pressure on Individual Sections psi.*	*Minimum Three-Edge-Bearing Load, lb. per linear ft.*
6	3/4	75	1100
8	7/8	65	1100
10	1	60	1250
12	1 1/8	55	1500
14	1 1/4	50	1600
15	1 3/8	50	1700
16	1 3/8	50	1800
18	1 3/4	50	2000
20	2	45	2100
21	2 1/4	45	2100
24	2 1/2	40	2400

Drainage Installation

Agriculture in Coachella Valley has a high per acre crop value as much of the farm land is in permanent plantings such as grapes, citrus and dates. Because of the high value right of way requirements are kept to a minimum width. Therefore, special equipment was necessary to do the construction. The drain tile is layed in an open trench excavated by a trenching machine, with a portable pipe laying crib, immediately behind the trenching machine. The drain tile is laid in a gravel filter to increase the drainage efficiently and also to keep the fine particles of soils from entering into and plugging the line. Backfilling immediately follows the pipe laying operation. This gives an installation of digging, laying and backfilling all in a single continuous operation. The pipe is laid to a depth that will give the farm drainage system a 7-foot outfall. As the District installs pipe up to 24 inches in diameter this means that the minimum depth of a 24 inch diameter pipe is 10 feet to the bottom of the ditch.

SUMMARY

(The program of drainage for the Coachella Valley was based upon the best basic data obtainable.) Attention was given to all phases of this problem; salinity content of drainage waters and soils was evaluated; soils were studied in relationship to the movement of water and engineering design was of the highest order.

Agricultural drainage is a field in which the civil engineer can use his technical knowledge and skill to make irrigation projects work for the service of mankind.

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PROGRESS ON BARRIER TO SEA WATER INTRUSION[a]

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The coastal portion of Los Angeles County has many groundwater basins. The pumping from these basins by water purveyors provides over 50 per cent of the local water demand. The supply to these groundwater basins comes mainly by valley infiltration of the runoff rising in the mountain watersheds. In their abilities to distribute water, the groundwater basins can be likened to pipelines where the water supply enters the upper end and portions of it are extracted as it moves along in the pipeline. If the demand on the pipeline is great, the head differential between the upper and lower ends of the pipeline must be large. Essentially, this is the case in the groundwater basins in Los Angeles County where the supply occurs on the upstream portions of the alluvial materials, and groundwater levels in the coastal plain are far below sea level - and have been for many years.

At the present time, sea water is intruding into the coastal groundwater basins in three large areas. The most extensive intrusion is occurring along the Santa Monica Bay coast line from Playa del Rey to the Palos Verdes Hills. Sea water intrusion also is occurring in the vicinity of the mouth of the San Gabriel River in the eastern part of Long Beach. In the area between Long Beach and the Palos Verdes Hills sea water intrusion has been detected in the shallow formations and may have started in the deep formations.

By 1950, sea water intrusion in the Manhattan Beach area had reached serious proportions, having caused the abandonment of seven producing wells. At that time the Los Angeles County Flood Control District conducted a recharge test in one of these abandoned wells using the balance of the wells for observation purposes. It was found in this test that it was feasible to inject water into a recharge well for an indefinite period providing that the injected water carried a high chlorine residual (in that test, not less than 10 ppm) and that the well be rehabilitated periodically.

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West Coast Basin Barrier Project

As a result of the successful recharge test conducted in 1950, a more extensive test was undertaken, with funds provided by the State of California, in the Manhattan Beach area beginning late in 1951 to determine the effectiveness of a barrier to sea water intrusion formed by injecting water into a line of wells parallel to the coast line. Previous papers have described this test in considerable detail. Briefly, the test facilities consisted of nine recharge wells spaced 500 feet apart along a line parallel to the coast, and 2000 feet inland. About 4.5 cfs were injected into these wells continuously, causing groundwater levels to rise above sea level and, thereby, present a barrier to sea water intrusion. Results of this test were observed in numerous observation wells drilled in the vicinity of the recharge line.

The test operation of these facilities was completed in 1954. Since that time, the wells have been operated as the initial portion of the West Coast Basin Barrier Project on a routine basis to provide a continuing barrier to sea water intrusion along its approximate 4500-foot reach. During this period, the injection of water has been discontinued only twice, each time for a period of about three weeks, for rehabilitation of sections of the Barrier Project supply pipeline.

In 1957, an experimental extension of the Barrier Project was undertaken. At that time, three additional recharge wells were installed to the north of the existing facilities and one to the south. On the basis of operating experience in the existing facilities and of exploratory geologic information obtained, spacing of the two most northerly wells was made 1000 feet rather than the initial test's 500 feet. The operation of the four new wells extended the length of protected coast line from 4500 feet to 7500 feet. Between February 1953, the beginning of injection, and December 1958, approximately 18,600 acre-feet of imported water have been injected through these facilities. This water has been traced 4000 feet inland through measurement of water quality in the various observation wells and by the plotting of isochlor maps.

Geologic Studies

Studies were begun early in 1957 to determine the facilities needed, and the costs involved in extending the Barrier Project to protect the entire 11-mile vulnerable reach along the Santa Monica Bay coast line downcoast of the Ballona escarpment.

The first step was to obtain more detailed geologic information on the area than was known from existing studies. In general, available data were sparse and at locations too distant from the coast line. At the Manhattan Beach test site, observation wells had been drilled mainly by the cable-tool method. This was due, largely, to the need to obtain good logs from the drilled wells and to have available adequate observation wells for recording and sampling purposes when drilling was completed. However, cable-tool holes for this purpose are relatively expensive, an average 300-foot 8-inch hole drilled for the District costing about $2600. For the new studies several cable-tool holes were drilled northerly of Manhattan Beach and one southerly of Manhattan Beach.

However, it was soon evident that a less expensive means was needed to obtain more geologic information, including data from greater depths. For this purpose, a program was begun of drilling exploratory wells with a rotary drilling rig. Geologic information from these holes was obtained through

careful selection of cuttings in the return mud, running electric logs on uncased mud-filled holes, and by correlating these electric logs, in a few wells, with information from nearby cable-tool holes.

Attempts were made to obtain groundwater data from these holes by placing 2-inch pipe, pre-perforated at desired levels, in the 5-inch rotary-drilled, mud-filled hole after the drilling and electric logging were completed. By a process of washing, air-jet pumping, and setting packers, most of the holes were made into adequate observation wells. In these wells, taped water levels can be obtained and water quality samples can be removed by air-jet pumping. No means has been devised of using a float-operated recorder on wells of this size.

One of the most significant factors determined in the geologic exploration was the delineation of multiple aquifers in a part of the area studied. Typically, the detritus consists of alternate layers of impermeable and permeable materials, usually three of each in the water-bearing zone.

The interbedding is important because it presents more complicated construction and operation problems for a Barrier Project. However, there is a compensating feature, in that the several aquifers will require a smaller total amount of water than would an aquifer of equal depth and transmissibility that was not multiple in nature. A full description of the reason for this feature would be too lengthy to discuss here, but it is related to the problem of balancing the pressure of sea water with a higher column of less dense fresh water.

In the area northerly of Manhattan Beach previous geologic information was generally confirmed inasmuch as the important water-bearing aquifers were found to be shallow and of relatively low transmissibility. However, it was found that over a portion of the area a "clay cap" does not exist. (To diverge, "clay cap" as used in this paper is a convenient term to denote an impermeable formation which is of sufficient areal extent to cause underlying permeable formations to have the character of pressure aquifers. The "clay cap" may be a member consisting of true clay, but is more likely to be silt or to be interbedded layers and lenses of materials of varying permeability.) The absence of a "clay cap" is important because in an aquifer where this member does not exist, part of the initial recharge water must be put into storage in the groundwater formation to provide the pressure needed to form a barrier. Although not lost, this extra water requirement is not desirable as it delays the creation of the barrier.

In the area southerly of Manhattan Beach, the geologic exploration proved to be very important. It was found that the important aquifers were not as deep, on the average, as they were expected to be. It was also found that the aquifers have some natural protection against sea water intrusion inasmuch as, in a portion of the area, a fault or fold was found parallel to and very near the coast line. One zone was found, which has particular significance to sea water intrusion. It is the Gardena Zone which is quite permeable, is normal to the coast line and is slightly less than a mile wide through the middle of Redondo Beach. It is probably a recent distributary of the Los Angeles River and, lying fairly near the ground surface, outcrops in the ocean close to the shoreline. It thus provides a ready conduit to carry sea water directly into the ground water basin. Sufficient evidence was derived to indicate that the deeper formations must outcrop on the ocean floor at considerable distances (say 3000 feet) from the shoreline. Information from oceanographic surveys of Santa Monica Bay showed that at these points of

outcrop a significant cover of relatively impermeable sea mud probably exists. It was found that a "clay cap" of sorts exists over the important aquifers throughout the southerly area except in the Gardena Zone. Of considerable importance to the determination of water requirements for the Barrier Project was the conclusion that the pattern of aquicludes and aquifers was very complex, that it included many lenses, that its layers thickened or thinned erratically, and that it phased into coarser or finer materials in short distances.

Engineering Studies

Several tools were used to evaluate the condition and properties of the underground formations, including the collection and analysis of information on ground water levels and ground water quality, the determination of transmissibility values of the formations, and the construction and operation of a field test facility.

Ground water contours, drawn from the data collected, showed general agreement with those previously drawn in the study area, except for two differences. One is an apparent anomaly in the ground water gradient near the coast line in the section in Redondo Beach. This factor tends to verify the geologic finding that the deeper aquifers outcrop at a considerable distance from the shoreline, and that the average inland gradient is actually somewhat shallower than previously thought. In addition, it was found that, in the multiple aquifer sections, each aquifer had a different pressure pattern with the deeper aquifers having the lower ground water levels.

Isochlor maps (lines of equal chloride concentrations) indicated the general progress of sea water intrusion inland from Santa Monica Bay. On the average, the 500 ppm contour had progressed 4000 feet inland northerly of Manhattan Beach and 7000 feet inland southerly of Manhattan Beach. The isochlor map shows the rapid travel of sea water in the Gardena Zone, where chloride concentration of 18,000 ppm has progressed 4500 feet inland from the coast line. It is evident from these maps that after the sea water enters through this permeable zone, it is fed into other zones with which it is in contact.

Determinations of transmissibility were made by running several field tests and by calculating, from well logs, comparative transmissibility values. Analysis of these values, along with results of tests by earlier investigators in the area, led to the conclusion that transmissibility values could be used only as a typical measure of short reaches of the aquifers.

A field recharge test was undertaken in Redondo Beach to obtain needed information on the deep and permeable aquifer encountered there. Several important facts were determined. Difficulties were found in the construction of the type of recharge well which had been drilled in Manhattan Beach because of problems encountered in drilling to the deeper depths at this location. At Manhattan Beach it had been found that 1 cfs was the nominal limit of injection in each recharge well; however, in the thicker and more permeable zone encountered at Redondo Beach considerably more water could be injected in each well, as much as 2 cfs over periods of several months. Recharge operations in Redondo Beach confirmed the suspicions of the geologists that the clay cap overlaying the Silverado formation was of questionable adequacy at this location; apparently there are enough permeable lenses or discontinuities in the fine material of the clay cap that it is not entirely effective under the influence of recharge operations.

Approximately 4 cfs were injected into two wells spaced 500 feet apart for a period of about two months. A relatively small mound was formed by this rate of injection, being on the order of four feet at the point midway between the wells. It is believed that results of this test are inconclusive insofar as determining the amount of water needed in this formation, because when utilizing only two wells the mathematical equations available do not evaluate properly the large end effects and the influence of the basin-wide pressure field.

The evaluation of the amount of water needed to maintain a barrier along the entire Santa Monica Bay coast line was based upon determining the inland flow of water in a number of short reaches using the relationship where the flow is proportional to the inland gradient, the cross-sectional width of aquifer, and the average transmissibility of the aquifer. It is estimated by this method that the amount of sea water moving inland from Santa Monica Bay amounts to about 50,000 acre-feet per year. The substitution of an approximately equal amount of fresh water, by injection through wells into the aquifers, will provide a barrier. The validity of the transmissibility method of determining water requirements was checked by evaluating the aggregate water supply and demand of the West Coast Ground Water Basin and was found to agree within 10 per cent.

Water Supply

The operating facilities at Manhattan Beach are presently using softened Colorado River water. The softened water has been treated sufficiently so that most materials which would have a clogging effect in recharge wells have been removed. However, adequate quantities of softened water are not available for expanded Barrier Project facilities in the present distribution system of the Metropolitan Water District. It appears that a supply of unsoftened water will be made available by the Metropolitan Water District in the vicinity of the City of El Segundo with adequate quantities of water to meet the requirements of the over-all West Coast Basin Barrier Project.

The District undertook a study to determine if untreated Colorado River water would be adequate for injection directly into recharge wells. Technical advice in this study was provided the District by Dr. Jack E. McKee. It was concluded in the study that it was quite unlikely that untreated water could be injected continuously in recharge wells without serious clogging and undue maintenance. It was a recommendation of the report that Colorado River water be filtered before it is injected into recharge wells. By a coincidence, other factors have occurred which occasioned the Metropolitan Water District to begin preparations for the filtering of the water to be supplied in the vicinity of El Segundo.

Proposed Barrier Facilities

The results of the geologic and engineering studies to date on the areas northerly and southerly of the existing facilities at Manhattan Beach are useful in studying the pros and cons of the several possible routes for the proposed barrier. In the area north of Manhattan Beach the proposed barrier probably would be between 1500 feet and 2400 feet from the coast line. Southerly of Manhattan Beach the proposed recharge line may progress gradually inland until at its maximum divergence from the coast line in Redondo Beach it would be 5200 feet from the ocean and would then bend

toward the coast line being 4200 feet from the ocean at its southerly terminus on the flank of the Palos Verdes Hills.

Present planning of the Metropolitan Water District calls for its pipeline to terminate at the easterly city limits of El Segundo. At this point, the Barrier Project's plan envisions carrying the water requirement for the northerly portion of the proposed barrier in a pipeline directly to the coast, and constructing a line for the southerly water requirement south and southwesterly to an intersection with the proposed recharge line in Redondo Beach. A varying-sized distribution pipeline would be used to carry the water along the line of recharge wells. The two proposed lines in combination with the proposed interconnected distribution pipelines and the existing feeder facilities would permit isolating sections of the line for maintenance purposes or emergency shutdown.

Just north of Manhattan Beach the proposed recharge line would traverse an area where there is no clay cap. Through this 12,000-foot zone it would be necessary to recharge under non-pressure conditions. Farther to the north, for about 10,000 feet, the recharge line again would enter an area where a clay cap exists and, in addition, the water requirement of the zone is considerably reduced. In the reach northerly of Manhattan Beach, recharge wells would be spaced from 1,000 to 2,000 feet apart. It is estimated that the water demand northerly of Manhattan Beach will be about 12 cfs.

Southerly of Manhattan Beach the proposed recharge line would traverse an area having a clay cap for about 9000 feet before it reaches the Gardena Zone. The Gardena Zone, as previously discussed, has no cap and is quite permeable. It is approximately 4500 feet in width measured along the proposed recharge line. In the 10,000 feet of proposed recharge line southerly of the Gardena Zone, a leaky cap appears to exist, as well as a multiple aquife condition. In the reach southerly of Manhattan Beach it is anticipated that wel spacing would vary from 750 feet to 250 feet, the latter in the zones with highest transmissibility. It is estimated that the water requirement in the area southerly of Manhattan Beach will be about 52 cfs, of which 16 cfs is in the reach traversed by the Gardena Zone.

The geologic and engineering information obtained and studied provides a basis of estimating the facilities required to maintain a fresh water barrier to sea water intrusion along the 11-mile coast line. It is estimated that, in addition to the 12 existing wells at Manhattan Beach, 60 to 65 recharge wells would be needed, along with about 180 observation wells to provide data to be used for barrier control purposes. The additional feeder pipelines and distribution pipelines along the recharge route would have a total length of about 88,000 feet and would vary from 54 inches to 8 inches in diameter. The preliminary estimated cost of installation of the additional facilities needed to complete the West Coast Basin Barrier Project is about five million dollars. This total includes the cost of installation of pipeline, drilling and equipping recharge and observation wells, and purchase and/or construction of appurtenant facilities.

Because of problems in financing, stage development of the balance of the barrier project is probable. From an engineering standpoint, it would be most desirable to expend initial efforts toward the control of the most critical locations. It is quite evident that the intrusion occurring in the Gardena Zone in the Redondo Beach area is the most critical to the welfare of the entire West Coast Basin. Another serious condition exists in the Manhattan Beach-El Segundo area, where a number of producing wells are close enough to the coast line so that sea water intrusion is an imminent problem.

Continuous operation of a fresh water barrier to sea water intrusion depends upon the continuous availability of a supply of fresh water. Although it is probable that the necessary supply will be available in the vicinity of El Segundo, there presently is no permanent legal entity which has the authority to buy the water needed. Industrious efforts are now under way in the coastal plain to form a water replenishment district, which will provide a permanent legal entity capable of such purchase. However, until a water supply is guaranteed by the formation of such a district, or by other means, expenditures for extensive construction of Barrier Project facilities would not be warranted.

In conclusion, it appears that a basis has been evolved for beginning the construction of the balance of the Barrier Project facilities needed along the Santa Monica Bay coast line. However, as in any project dealing with the conditions to be found under the earth's surface, each new step must be undertaken only after analyzing the results of the last. It is quite likely that, before the project is completed, the preliminary plan presented herein will be modified in many ways.

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DRAINAGE PROBLEMS OF THE SAN JOAQUIN VALLEY

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ABSTRACT

This paper outlines the geography and geology of the San Joaquin Valley of California, with special emphasis on the factors affecting drainage. Following this, the drainage problems as they exist today are unfolded through a description of the cultural changes that have brought them into sharp focus; and, finally, the objectives and scope of the drainage investigation currently in progress are described.

INTRODUCTION

The San Joaquin Valley of California is a vast and rich agricultural area, containing more than 7,000,000 acres of potentially irrigable valley floor lands. Irrigation was initiated in the 1850's and involved some 800,000 acres by 1900. From that point forward development was very rapid, resulting in an irrigated acreage of about two and one-quarter million acres by 1930, and approximately 4,000,000 acres today. This development has involved many profound changes in the water resources of the valley.

Irrigation engineers have diverted the flow of numerous rivers and imported water hundreds of miles from others. Large natural lakes have evaporated or have been drained. The normal seasonal distribution of runoff has been altered by detaining flood flows in foothill reservoirs for later release during the low flow period of the year. As a result of such developments, oftentimes uncoordinated, many serious problems have occurred. It

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is a credit to the people of the San Joaquin Valley, and to the others who have worked with them, that many of these problems have already been solved. Probably the most perplexing of the remaining and unsolved problems are those of drainage. This paper discusses the drainage problems of the San Joaquin Valley and describes studies which the California State Department of Water Resources is directing toward their eventual solution.

A complete and comprehensive discussion of the drainage problems of the San Joaquin Valley is beyond the scope of this paper, involving as it would highly technical areas in the fields of soil physics, chemistry, hydraulics, and botany, to mention only a few. The particular drainage problem discussed in this paper is one which may be termed the disposal problem. Our current studies of disposal of drainage waters in the San Joaquin Valley include determination of the drainage relationships between various large areas within the valley, determination of requirements for drainage from each of these areas, and the design of facilities to remove the determined quantities of drainage water where the need exists.

Geography

The San Joaquin Valley, drained by the San Joaquin River, comprises the southern and larger portion of the great Central Valley of California. The floor of the Central Valley extends through five and one-half degrees of latitude, and attains a maximum width in excess of 60 miles. On the north it is bounded by the Cascade Range from which the Sacramento River issues, while the Tehachapi Mountains form the southern boundary. The crest of the massive Sierra Nevada forms the eastern edge of the valley, and the Coast Range separates it from the Pacific Ocean. The northern portion is drained by the Sacramento River and its tributaries and is called the Sacramento Valley. The waters of the Sacramento and San Joaquin Rivers join a delta region, enter Suisun Bay, and then flow into San Francisco Bay through the Carquinez Straits, providing the only natural outlet from the sea. The geographic features are shown in a generalized way in Figure 1.

Referring to Figure 1, it may be noted that the San Joaquin Valley is actually composed of two separate basins; the upper basin to the south, known as the Tulare Lake Basin; and the lower or northern portion of the valley known as the San Joaquin River Basin. Figure 2 shows in somewhat greater detail the location of the principal streams of the San Joaquin Valley as well as showing the location of several cities and towns of the area.

Tulare Lake Basin

For all practical purposes, the Tulare Lake Basin is a closed basin draining into Tulare Lake. It contains approximately one-half of the irrigable valley floor lands of the San Joaquin Valley and receives about one-quarter of the valley's total available runoff. As an indication of water supply conditions, an average annual depth of precipitation of less than five inches falls on much of the valley floor in this arid region, while annual runoff from the adjacent mountains is less than one acre-foot per acre of irrigable land.

Historical changes in the configuration of Tulare Lake have had considerable bearing on the present drainage problems of the Tulare Lake Basin. At one time the Kings River, in the northern portion of the basin, and probably all other streams that are now tributary to Tulare Lake, were a part of

FIGURE 1
STATE OF CALIFORNIA
MAJOR DRAINAGE SYSTEMS
OF SAN JOAQUIN VALLEY

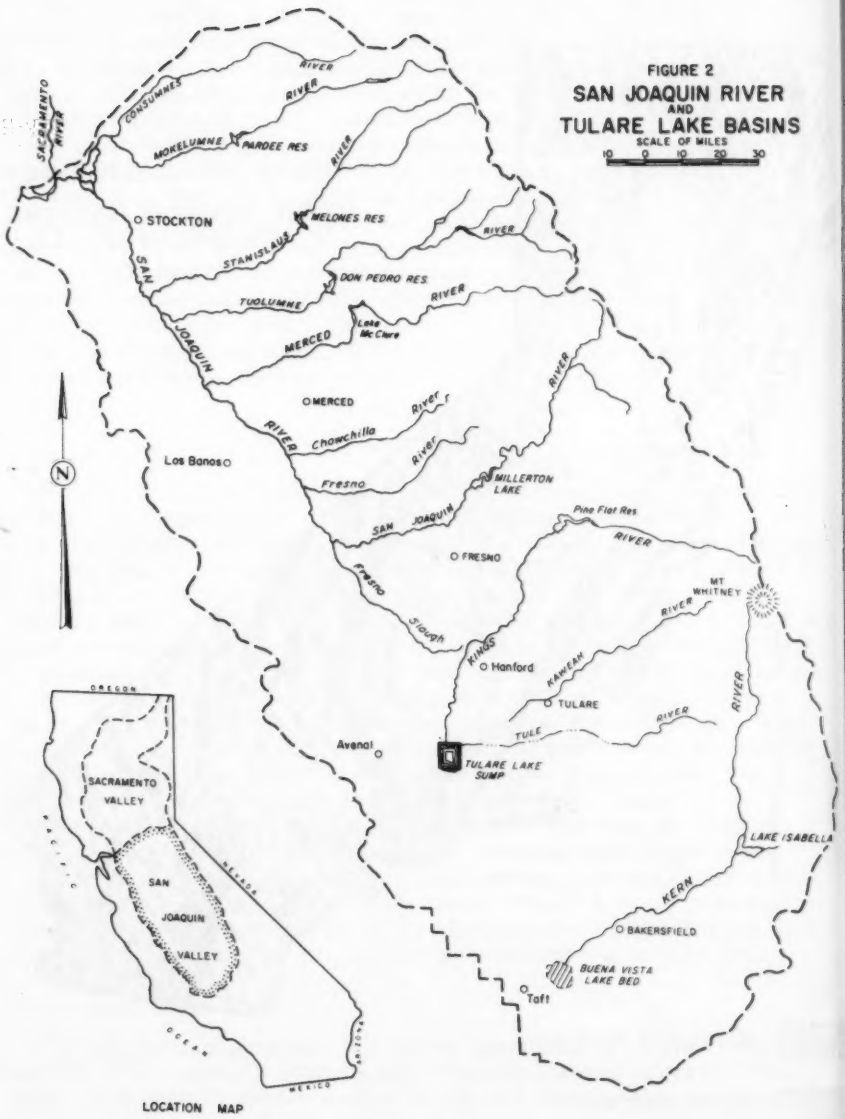

FIGURE 2
SAN JOAQUIN RIVER
AND
TULARE LAKE BASINS
SCALE OF MILES
10 0 10 20 30

the San Joaquin River system that now drains only the San Joaquin River Basin. However, the reduction in gradient of the Kings River on reaching the valley floor caused deposition of large quantities of material, eventually forming a delta which created a barrier to outflow from the Tulare Lake Basin. Tulare Lake, which was so formed, extended over an area of 850 square miles at maximum stage when it spilled to the north. However, the lake has failed to spill for a great number of years, due to high evaporative losses during the long hot and dry summers and the extensive diversion of tributary stream flow to higher lying lands. Recently constructed flood control dams at the foothill line have caused nearly complete disappearance of the lake. Today only a few square miles are subject to inundation by uncontrolled winter flows, and much of the old lake bed is extensively farmed.

A similar situation occurs at the southerly end of the Tulare Lake Basin where the Kern River reaches the trough of the valley; and the history of recession of Buena Vista Lake has followed much the same pattern as Tulare Lake, although the area involved is much less, amounting to only about 40 square miles.

San Joaquin River Basin

The San Joaquin River Basin consists of all lands drained by the San Joaquin River and derives its name therefrom. This basin enjoys the advantage of direct drainage to the sea by way of San Francisco Bay, and experiences somewhat greater precipitation than does the Tulare Lake Basin. The annual depth of precipitation over the valley floor averages about 12 inches, while runoff from the tributary mountainous areas averages some two acre-feet per irrigable acre.

A geographical feature of significance to drainage problems of the San Joaquin River Basin is the extensive area once subject to annual overflow along the rivers, some portions of which even today are subject to occasional overflow. These lands have been historically devoted to pasture and quick-growing crops, with irrigation accomplished through natural or intentional flooding only during periods of high flow. Some of these lands are readily reclaimable through ordinary drainage and leaching techniques; but many have a high concentration of exchangeable sodium, and reclamation requires the addition of soil amendments and other special measures.

Geology

The San Joaquin Valley is a structural trough filled with sediments to depths varying from a few thousand feet at the northern end to something in excess of 30,000 feet in the southern portion. Generally speaking, the sediments at depth are of marine origin and contain highly saline connate waters. The sediments forming the upper zone are of continental origin and contain the usable ground water of the valley. The continental deposits are largely of river origin, with the discontinuity and heterogeneity associated with this type of deposition. However, there are significant continuous and homogeneous local deposits of lake origin; and a bed of lake deposited diatomaceous clay, with a thickness of as much as 150 feet, apparently continuously underlies approximately 5,000 square miles of the valley. This deposit, known as the Corcoran Clay, forms an effective barrier to the vertical movement of water.

The geology of the valley is thus characterized by three generally distinct zones of ground water storage, a condition which occupies a very important place in any study of drainage problems and removal of drainage waters. First, at depth we find the connate zone, containing water of sea-like constituency. Second, there is an extensive zone of confined ground water beneath the impermeable Corcoran clay. Finally, there is the free ground water zone overlying the Corcoran clay. It should be emphasized that this general geologic situation considerably complicates the drainage problems.

Irrigation and Drainage

When irrigated agriculture first began to develop in the San Joaquin Valley, there existed along the east wide a vast area of higher lying valley lands, fed by apparently adequate streams of high quality water. At the same time a marsh, 5 to 50 or more miles wide, occupied the central trough of the valley throughout its length, even giving way to lakes at several points. The area west of the trough in the Tulare Lake Basin was a barren desert for the most part, while the west side of the San Joaquin Basin was suitable primarily for stock grazing purposes because of the nonexistence of live streams from which to make irrigation diversions. The major irrigation problem in the valley at that time was one of regulating the extreme seasonal fluctuation of runoff which is so typical of California streams.

Tulare Lake Basin

Irrigation in the San Joaquin Valley was first practiced adjacent to the Kings River and other east side streams of the Tulare Lake Basin. The developments consisted of diversion of stream flow on a catch-as-catch-can basis with little or no regulatory storage. Water application practices were rather careless, but nevertheless the benefits of irrigation were clearly apparent. Developments based on use of ground water were initiated following appropriation and use of the readily available portion of the surface supply.

In much of the Tulare Lake Basin, ground water was originally obtained by simply drilling a well, as artesian conditions largely prevailed. However, as development continued and water levels were drawn down, the installation of pumps became necessary. Returns from investments in irrigation were so attractive that the ground water was extensively developed, making use of increasingly deeper wells. Today, it is not unusual to find wells 2,500 feet deep with pumping lifts of four to six hundred feet.

In the areas where the deeper wells have been drilled the supply is being drawn from beneath the Corcoran clay. Withdrawal from this confined zone far exceeds natural replenishment, and there is generally no possibility for percolation of excess applications of irrigation water. Consequently, there exists a paradoxical situation wherein the ground water supply is being exhausted, and yet the danger of buildup of a high water table is very real, having, in fact, occurred in some areas. In many cases the zone above the Corcoran clay can be depended upon to yield large enough quantities of water to permit drainage control through pumping. However, this solution of the drainage problem cannot be generally utilized, since the poor mineral quality of the water so obtained may preclude re-use. In any event, such re-use could only be a stop-gap solution, since it would result eventually in intolerable concentrations of minerals in the upper zone. The drainage disposal

problem, coupled with the high operating costs associated with ground water pumping, has inspired an unusually high irrigation efficiency in many instances, with little or no percolation through the root zone. This practice is causing higher and higher concentrations of minerals in the upper few feet of soil, and can be expected to seriously impair agricultural productivity in the future, if not corrected.

In the portion of the Tulare Lake Basin that is not underlain generally by a confining clay, the drainage problem is less pressing insofar as presently developed areas are concerned. However, as water is applied to newly developed lands it has the effect of introducing back into the irrigation cycle all the soluble salts that have been deposited during the centuries that water has been evaporating from the land surface. Although no widespread effect on quality in the ground water basin has as yet been observed, considerable local concern has been expressed because of the increasingly higher mineral concentrations occurring in the return flow, which is generally reapplied lower in the basin.

In reviewing the situation in the Tulare Lake Basin, it becomes clear that there are three basic interrelated factors that affect drainage requirements. The first of these is that more water is presently consumptively used in the basin than flows into it. A number of projects have been studied to remedy this condition, including various proposals for importation of supplemental water from distant sources. The most feasible projects are those that propose the pumped diversion of surplus flows in the Delta of the Sacramento and San Joaquin Rivers.

The second basic factor is the excess of salt inflow to the Tulare Lake Basin over the salt outflow. The projects proposed to solve the problem of overdraft will tend to some extent to aggravate this condition. This is true because of the higher mineral concentration in supplemental water to be imported from the Delta than is found in natural local supplies in the basin. This problem has been recognized by the State in studies made to evaluate the supplemental requirements of the Tulare Lake Basin; and the estimates of water application requirements have been increased to provide for salt balance and to maintain favorable limits to concentration of minerals in the root zone.

The third basic factor affecting drainage requirements in the Tulare Lake Basin, and the one that is probably most significant at the present time, is the leaching action resulting from irrigation of previously undeveloped lands that have been accumulating salts for thousands of years. This will be recognized as a typical drainage problem associated with irrigation reclamation activities in arid regions. The usual solution to such a problem is to provide drainage facilities extending to the nearest major waterway to the sea. In the case of the Tulare Lake Basin the nearest such stream is the San Joaquin River. Unfortunately, the San Joaquin Basin is also experiencing drainage troubles, and the disposal of additional drainage water of poor mineral quality into the San Joaquin River would compound an already difficult problem.

San Joaquin River Basin

The relation of the development of the water and land resources to the drainage problem in the San Joaquin River Basin is somewhat more complex than in the Tulare Lake Basin. The San Joaquin River flows in a westerly

direction to the trough of the valley, then turns sharply northward to constitute the main stream of the valley until it joins the Sacramento River and enters San Francisco Bay. The river is joined at frequent intervals by major tributaries from the Sierra Nevada to the east, but with few and meager tributaries from the arid west side.

Early irrigation development consisted of diversions from the San Joaquin River in the vicinity of the point where it swings northward. These diversions were utilized primarily for irrigation of adjacent lower lying lands by wild flooding, in order to increase the production of natural forage crops and hay. Although this development somewhat increased the consumptive use of water in the basin, it had little effect on over-all drainage conditions.

With the passage of time, however, the advantages of irrigating arable lands of the San Joaquin River Basin became obvious and many irrigation companies and districts were formed. Diversions were made from tributary streams, and not only were the natural summer flows depleted, but regulatory storage was developed in the foothills to permit summer use of a portion of the winter flows. This storage reduced the threat of flooding of the lower lying lands, and thereby encouraged higher uses and additional reclamation in the trough area. Hydroelectric power facilities, involving additional storage, were constructed in the mountainous areas of the basin, with further regulatory effect on the regimen of stream flow.

The rapid growth of the metropolitan area surrounding San Francisco Bay has led to the storage and export of significant quantities of high-quality water from streams of the San Joaquin River Basin. The Central Valley Project has resulted in the export of high quality water from the San Joaquin River to the Tulare Lake Basin, but, more significantly, it has resulted in the importation of water containing greater quantities of dissolved minerals than does water native to the basin.

Ground water extractions within organized districts in the San Joaquin River Basin are generally made primarily for drainage control, although ground water constitutes the principal source of supply for many lands in non-organized areas and for local municipalities. The availability of a relatively plentiful and inexpensive surface water supply in many areas, and the poor quality of the ground water in other areas, have resulted in less extensive development of ground water in the San Joaquin River Basin than has occurred in the Tulare Lake Basin. As a consequence, the basin-wide picture is one of rising ground water levels, as would be expected in an area served primarily from surface sources. However, there are local instances of the lowering of ground water levels associated with overdraft.

Some of the more significant developments that have been described for the San Joaquin Basin have occurred in the past decade, and the effects have not been properly evaluated as yet. The basic problem, however, is quite apparent. Progressive depletion of stream flow has occurred due to both upstream beneficial use and export from the basin. This has apparently been accompanied by increased mineral concentrations in accretions of water to the valley floor during the irrigation season, to such an extent that diverters from the main stream are justifiably concerned with protection of the quality of their supply.

Planning to Solve the Drainage Problems

Recent water project planning for the San Joaquin Valley has taken cognizance of the importance of drainage disposal. The necessity to maintain root

zone salinity below an acceptable level has been acknowledged, and has served as a consideration in the evaluation of project water requirements.

The California Water Plan, which has been prepared by the Department of Water Resources as a comprehensive guide to future development and utilization of the water resources of the State, includes a finding that a master drain will be necessary ultimately if the potential productivity of the San Joaquin Valley is to be realized. In June, 1955, several irrigation agencies dependent for their supply upon the San Joaquin River entered into an agreement with the State providing for an evaluation of present and future quality of water in the lower reach of the river from which they divert. Finally, the Joint Committee on Water Problems of the California Legislature investigated the drainage problem of the San Joaquin Valley. As a result of recommendations published in this committee's report, the San Joaquin Valley Drainage Investigation was initiated by the State Department of Water Resources in June of 1957.

The San Joaquin Valley Drainage Investigation, which has been programmed to extend over a four-year period, has two broad objectives: (1) formulation of a detailed master drainage plan for the entire San Joaquin Valley, and (2) intensive study of areas in present urgent need of drainage, including evaluation of the problems, design of works needed to solve the problems, and all other studies necessary to determine the feasibility of construction of the drainage facilities.

To attain the first objective it is necessary to determine the drainage requirements of all areas of the valley under present and forecast future conditions of development. To do this we are largely utilizing a companion study by the Department to determine the rate of growth of economic demands for supplemental water. This forecast is based on classification of lands, selection of a crop pattern consonant with the land capability, cost of water, and estimates of future market conditions. With this estimated rate of development as a guide, we then will determine the drainage requirements, staged with respect to time for each area. This will place us in position to prepare a detailed physical plan for achieving basin-wide drainage, making use of essentially the same techniques as are used to devise a plan for basin-wide distribution of water.

The second broad objective of our investigation is the development of projects for immediate construction where necessary to solve existing urgent problems of drainage disposal. These projects are to be such that they can be integrated to the fullest practicable extent into the master plan. In this case we are dealing with presently developed areas, and can measure many of the factors affecting drainage.

Determination of the justification for construction of drainage facilities will be accomplished by comparisons of estimated benefits to the area, both with and without project facilities, and also with various proposed facilities in operation. These studies will of necessity include consideration of extensive land areas, since the benefit to be derived by providing an outlet for drainage water of poor mineral quality can accrue to lands entirely removed from the area drained.

Finally the financial feasibility of the selected project will be determined. This determination will consist primarily of an exploration of the ability and willingness of all proposed participants in the project to pay their allocated costs.

Based upon our experience here in California, it is apparent that in planning for future irrigation projects, we must consider the so necessary attendant drainage facilities; and that in many cases the costs of the drainage facilities should be included in the determination of economic justification and financial feasibility of the project as a whole. As you who are associated with irrigation know, irrigated lands that today are highly productive and capable of supporting a substantial outlay to maintain their productivity can in a few years be reduced to an alkaline waste if proper drainage facilities are not provided.

Journal of the

IRRIGATION AND DRAINAGE DIVISION

Proceedings of the American Society of Civil Engineers

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Note: Paper 2179 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 85, IR 3, September, 1959.

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MONTHLY CONSUMPTIVE USE REQUIREMENTS FOR IRRIGATED CROPS[a]

Discussion by N. A. Halkias

N. A. HALKIAS.[1]—Mr. Blaney's paper presents a summary of measured monthly rates of consumptive use of water for different irrigated crops in Western United States. The data presented by Blaney and Criddle (1950)(1) indicate that the Blaney - Criddle formula was developed principally to give the seasonal consumptive use of water.

Halkias, et al.(5) in an analysis of the actual measurements of the use of water by crops at Davis, California, together with the use of the Blaney - Criddle method found that for the most critical month of the growing season for consumptive use, July, the Blaney Criddle method gave values 14-38 per cent lower than the measured values.

In this analysis, the seasonal consumptive use coefficients have been used for determining the monthly consumptive use of water for July.

Data presented by Criddle 1958,(3) appendix table 3,1 indicate that the maximum monthly coefficient (k) for the more important irrigated crops in the West, is 25-60 per cent higher than the seasonal consumptive use (K).

Likewise, data presented by Blaney 1959,(2) table 7, indicate that the monthly coefficient (k) for alfalfa ranges from 0.74 to 1.36 for summer months.

All data mentioned above result that the monthly coefficient (k) in the formula U = K F, differs from month to month during the summer period and that it may be 60 per cent higher than the seasonal consumptive use coefficient (K).

In designing irrigation systems and in planning irrigation programs, it is necessary to determine the water use by crops for the most critical month of the growing season. In this case the Blaney-Criddle method give us little information.

On the other hand Table 1 resulting from data presented by Halkias et al. in Hilgardia,(5) shows that the monthly coefficient S in the formula U = SD of the atmometer method is practically the same for each one of the summer months. The atmometer method, consequently, gives the best possibilities for determining the monthly water use by irrigated crops.

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a. Proc. Paper 1963, March, 1959, by Harry F. Blaney.

1. Irrig. Specialist, Ministry of Agriculture, Greece.

TABLE 1

Measured Monthly Use of Water by Crops, Monthly Difference in Evaporation Between Black and White Atmometers and Monthly Coefficient S in the Formula U = SD for Ten Different Crops in California

Crops and Month	Measured Use of Water U Inches	Difference Between Atmometers D c.c.	Monthly Coefficient S	Mean Coefficient or Slope of the Regression Line	Per Cent Difference Between Monthly and Mean Coefficients
Alfalfa (1937-38)					
May	6.8	532	0.0128		
June	7.9	597	0.0132		
July	8.3	609	0.0136	0.0134	1.5
August	7.1	526	0.0135		
September	4.3	320	0.0134		
Sugar Beets (1933-35)					
May	5.2	535	0.0097		
June	5.7	576	0.0099		
July	7.1	759	0.0094	0.0096	2.1
August	5.8	614	0.0094		
September	-	-	-		
Apricots (1954)					
May	-	-	-		
June	5.6	510	0.0110		
July	6.8	550	0.0124	0.0120	3.2
August	6.5	520	0.0125		
September	4.9	400	0.0123		
Tomatoes (1933-35)					
May	...	...	...		
June	3.2	390*	0.0082		
July	6.2	759	0.0082	0.0082	0.0
August	4.9	614	0.0080		
September	4.7	547	0.0086		
Peaches (1931-38)					
May	5.4	530	0.0102		
June	6.4	578	0.0111		
July	7.9	691	0.0114	0.0110	3.5
August	7.2	645	0.0112		
September	5.0	547	0.0101		
Walnuts (1935-38)					
May	6.4	493	0.0130		
June	6.7	530	0.0126		
July	8.4	569	0.0147	0.0135	8.1
August	7.2	533	0.0135		
September	4.8	402	0.0119		
Prunes (1934-38)					
May	5.8	547	0.0106		
June	6.0	571	0.0105		
July	7.6	700	0.0109	0.0108	0.9
August	6.5	592	0.0110		
September	5.0	468	0.0107		
Cotton (1947)					
May	...	...	...		
June	6.9	720	0.0096		
July	7.4	714	0.0104	0.0105	1,0

*This value is for the period June 12 to June 30.

TABLE 1
(continued)

Crops and Month	Measured Use of Water U Inches	Difference Between Atmometers D c.c.	Monthly Coefficient S	Mean Coefficient or Slope of the Regression Line	Per Cent Difference Between Monthly and Mean Coefficients
August	6.0	582	0.0103		
September	5.0	485	0.0103		
Grapes (1933-35)					
May	4.6	535	0.0086		
June	4.9	576	0.0085		
July	6.2	759	0.0082	0.0086	4.9
August	5.3	614	0.0086		
September	4.3	547	0.0079		
Artichokes (1939)					
May	3.2	440	0.0073		
June	3.2	433	0.0074		
July	2.8	374	0.0075	0.0073	2.7
August	2.3	325	0.0071		
September	2.2	318	0.0069		

2. Blaney, Harry F., Monthly Consumptive Use Requirements for Irrigated Crops, Journal of the Irrigation and Drainage Division Proc. of the ASCE, March 1959.

3. Criddle, Wayne D., Methods of Computing Consumptive Use of Water, Journal of the Irrigation and Drainage Division, Proc. of the ASCE, January, 1958.

4. Goodrich, R. D., Discussion of "Methods of Determining Consumptive Use of Water in Irrigation". Journal of the Irrigation and Drainage Division, Proc. of the ASCE, May, 1957.

5. Halkias, N. A., Veigmeyer, F. J. and Hendrickson A. H., Determining Water Needs for Crops from Climatic Data, Hilgardia, December, 1955.

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DRAINAGE OF AGRICULTURAL LANDS USING INTERCEPTOR LINES[a]

Discussion by David K. Todd and Jacob Bear

DAVID K. TODD,[1] M. ASCE and JACOB BEAR.[2]—This paper treats the problem of a horizontal drain intercepting unconfined ground water flowing over a sloping impermeable bed. Although Eq. (5) of the paper is approximate, it is of practical value for estimating the drawdown upslope from the drain, as Mr. Donnan points out. It is interesting to note that this equation, attributed to unpublished writings of R. E. Glover, was derived in an identical manner in a Swiss hydraulics textbook by Jaeger(1) in 1949. An English translation was published in New York in 1957.(2) The only difference in the two derivations is that Jaeger defined the y-coordinate as vertical (see Fig. 1-a) rather than perpendicular to the bed (Fig. 1-b). For small slopes to which the problem applies in the field, the difference of the cosine of the slope is negligible, hence the resulting drawdown equations of Jaeger and Glover are identical.

Neither the Jaeger nor the Glover coordinate systems are quite satisfactory. The Jaeger coordinates (Fig. 1-a) involve a slope measurement and are not perpendicular. The Glover coordinates (Fig. 1-b) both involve slope distances. It is customary to think of an x-y coordinate system as representing horizontal and vertical directions. For the present application field distances are measured horizontally and depths are measured vertically. Based upon this usual coordinate system, shown in Fig. 1-c, the Jaeger-Glover equation can be derived as outlined in the following paragraph.

With the Dupuit assumptions and assuming S is small, then Darcy's law describing the flow in Fig. 1-c can be written

$$Q = K(y - Sx)\frac{dy}{dx} \tag{1}$$

Rearranging gives the differential equation

$$\frac{dx}{dy} + \frac{KS}{Q}x = \frac{K}{Q}y \tag{2}$$

which has for its solution

$$x = \frac{1}{S}\left[-\left(h - \frac{Q}{KS}\right)e^{\frac{SK}{Q}(h-y)} + \left(y - \frac{Q}{KS}\right)\right] \tag{3}$$

Substituting the upstream condition of flow

$$Q = KHS \tag{4}$$

a. Proc. Paper 1964, March, 1959, by William W. Donnan.
1. Associate Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.
2. Grad. Research Engr., Univ. of California, Berkeley, Calif.

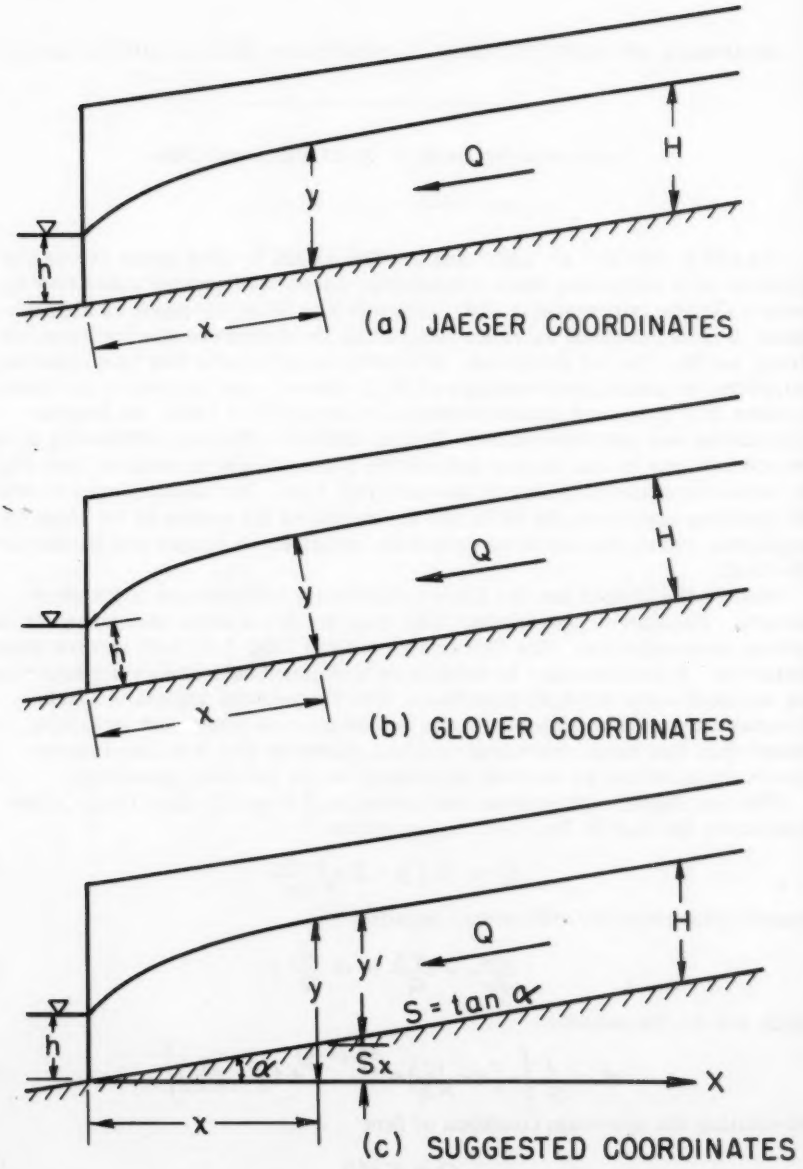

FIG. I COORDINATE SYSTEMS FOR THE UPSLOPE DRAWDOWN EQUATION

in Eq. (3) yields

$$x = \frac{1}{S}\left[-(h-H)e^{\frac{h-y}{H}} + (y-H)\right] \tag{5}$$

Now, by redefining the flow depth as

$$y' = y - Sx \tag{6}$$

and substituting this in Eq. (5) gives

$$x = \frac{1}{S}\left[H \ln \frac{H-h}{H-y'} - (y'-h)\right] \tag{7}$$

which is the Jaeger-Glover equation.

The author quotes Childs' experimental work(3) which led to the conclusion that the upslope influence is inversely proportional to the gradient of the bed. The Jaeger-Glover equation states this same relationship independent of H and h values. Therefore, it is not clear why the author states that the Childs' inverse relation "does not hold under varying conditions of depth of saturated flow and depth of interception."

For practical applications by drainage engineers, the Jaeger-Glover equation is modified by Mr. Donnan to let y = 0.9H, thereby securing finite values of x where drawdown is significant. Although x approaches infinity as y approaches H, it should be noted that the distance to a significant drawdown also approaches infinity as S approaches zero. Thus, for the limiting condition of a horizontal bed, the drawdown influence is theoretically infinite in extent.

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UPSTREAM IRRIGATION IMPACT ON COLUMBIA RIVER FLOWS[a]

Discussion by J. Ernest Flack

J. ERNEST FLACK,[1] A. M. ASCE.—The author's paper is most worthwhile in pointing out some of the benefits of upstream irrigation in reducing peak flows and increasing low flows at downstream locations. An additional benefit of upstream uses (that include transbasin diversions as well as irrigation and storage) is that of water salvage attributable to reduction of flood peaks. This characteristic is probably significant on streams that exhibit considerable overbank flow during floods.

Studies of apparent salvage water attributable to transmountain diversions were reported by the Upper Colorado River Compact Commission in their Fourth Annual Report.(1) These studies covered the Upper Colorado River upstream from Glenwood Springs and Hot Sulphur Springs. At the suggestion of Mr. R. M. Gildersleeve, Chief Engineer of the Colorado Water Conservation Board, the writer reviewed and extended these studies.(2) A brief outline of the methods used in this latter study are presented as an addition to the author's "Modifications of Flow".

The general procedure was the use of the Inflow-Outflow method. Monthly flows at designated Inflow stations were corrected for changes in storage and diversions and then correlated against flow at a designated Outflow station. A significant selection of Inflow stations' flows were summed and plotted by months against the Outflow station's flow. An approximate straight line correlation was made for the early period of record, when upstream diversions and storage were insignificant. Since then, there has been a graduate increase in upstream depletions almost entirely due to storage and diversion by transmountain diversion projects. Marked increases occurred more recently after completion of major transmountain diversion projects such as the Colorado-Big Thompson Project.

In order to illustrate the method more clearly, reference is made to Fig. 1. This is a schematic representation showing the straight line correlation of Inflow versus Outflow as determined for an early period when upstream depletions and storage were insignificant. The point marked A represents the correlation between Inflow and Outflow as recorded for a month when upstream depletions were significant. The shift of the Outflow value to point A' represents the correction for upstream depletions. The amount of apparent salvage water is the distance the point A' falls to the right of the straight line correlation.

Because of a lack of long time records at some Inflow stations, monthly correlations were not always satisfactory. For this reason double mass curve

a. Proc. Paper 1965, March, 1959, by Harold T. Nelson.

1. Asst. Prof., Dept. of Civ. Eng., Univ. of Colorado, Boulder, Colo.

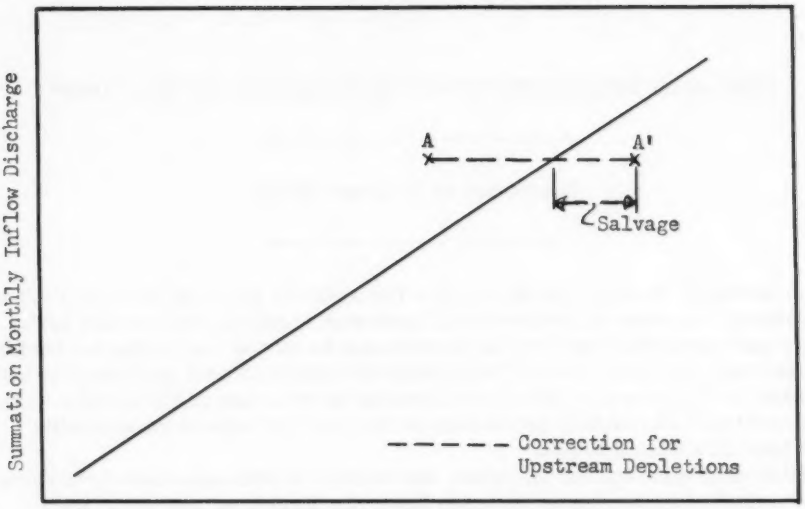

Figure 1 Salvage From Inflow-Outflow Correlation

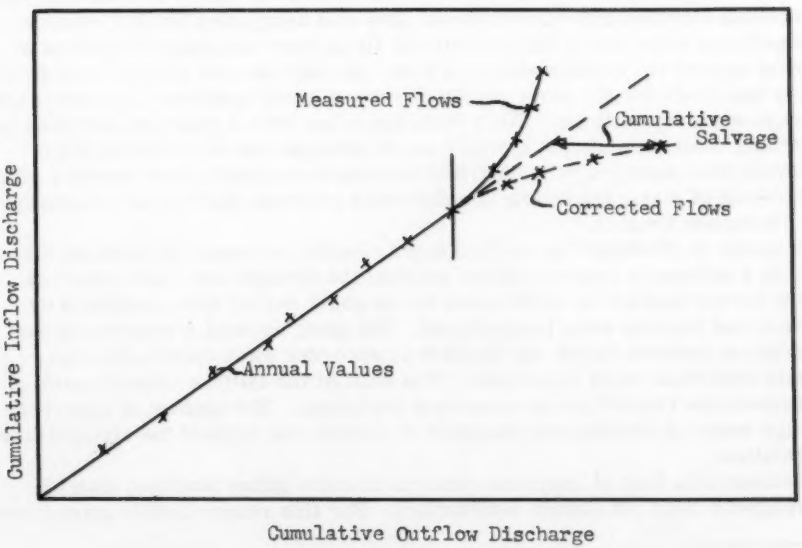

Figure 2 Salvage From Cumulative Discharge Mass Curve

plotting of Summation of Inflow versus Outflow on a successive water year basis was used, as shown schematically, in Fig. 2. For the selected stations a straight line correlation was determined for early period before significant transmountain diversions were made. For the following periods, as upstream depletions increased significantly, the departure of the measured Outflow from the straight line correlation is indicated by the upward swing of the solid line on Fig. 2. The Outflow corrected for upstream depletions is shown as a dashed line curving downward and the departure of this dashed line from the straight line correlation is a measure of the water salvaged.

The following tabulation gives some of the results of a study using the latter of the above methods for the Colorado River above Hot Sulphur Springs.(2)

OUTFLOW STATION - COLORADO RIVER AT HOT SULPHUR SPRINGS

Period	Average Upstream Depletions acre ft/yr	Average Apparent Salvage acre ft/yr
1936-49	50,100	28,400
1950-54	231,900	33,700

These results are only approximate because corrections for bank storage and evaporation in newly constructed reservoirs have not been made.

It is believed that on streams which have considerable losses during floods or spring snow-melt periods from overbank flow that reduction of the peaks will show significant salvage benefits. A brief summarization of the benefits of upstream depletions by irrigation, storage, and diversions could then be:

(1) Increase of low seasonal flows.
(2) Reduction of flood peaks.
(3) Salvage due to reduction of losses.
(4) Regulation by underground storage.

As the author has so aptly pointed out, the upstream irrigator has not depended on downstream benefits in the financing of irrigation projects, but now and in the future "the identification, evaluation, and allocation of downstream benefits from upstream river regulation" will be most necessary if upstream projects are to progress at a reasonable rate to their ultimate development in the face of conflicting demands by downstream water users.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (CP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758 (ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767 (WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

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DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CC1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

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AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

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c. Discussion of several papers, grouped by divisions.

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IR3

SEPTEMBER 1959 — 34 NO. IR 3
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE IRRIGATION AND DRAINAGE DIVISION OF ASCE

JOURNAL OF THE IRRIGATION AND DRAINAGE DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

IRRIGATION AND DRAINAGE DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

September, 1959

EXECUTIVE COMMITTEE, IRRIGATION AND DRAINAGE DIVISION, ASCE

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William W. Donnan, Secretary
P.O. Box 629, Pomona, Calif.

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7024 Melrose Avenue, Los Angles 38, Calif.

N. A. Christensen, (Cornell University)
412 Haushaw Road, Ithaca, New York.

Finley Laverty, Contact Member from Board of Direction
P.O. Box 2418, Terminal Annex, Los Angeles, Calif.

* * *

Carl R. Wilder, Newsletter Editor
3223 So. Columbine St., Denver 10, Colo.

CALENDAR OF COMING MEETINGS

October 19-23, 1959. National Convention, ASCE, Hotel Statler, Washington, D. C. Irrigation and Drainage Division plans to sponsor three half-day sessions on the theme, "World Wide Picture of Irrigation."

October 27-30, 1959. Annual Convention, National Reclamation Association, Shirley Savoy Hotel, Denver, Colorado.

March 7-11, 1960. National Convention, ASCE, Jung Hotel, New Orleans, La. Irrigation and Drainage Division plans to hold three half-day sessions as

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follows: Irrigation and Drainage in Humid areas; Humid area water rights; Drainage.

June 19-23, 1960. National Convention, ASCE, Reno, Nevada, Irrigation and Drainage Division tentatively plans to hold four half-day sessions on themes particularly a propos in the western irrigated states.

Get in touch with one of the following members of our committee on Sessions Programs if you would like to offer a paper or discussion at any of these or other I. & D. programs.

Kenneth Q. Volk, Chairman, 7024 Melrose Ave., Los Angeles, Calif.
N. A. Christensen, 712 Haushaw Road, Ithaca, N. Y.
George D. Clyde, 1747 Browning Ave., Salt Lake City, Utah.
O. W. Israelsen, 485 East 6th North St., Logan, Utah
Herbert E. Prater, 2001 So. Madison St., Denver 10, Colo.

Chairmen of other standing committees of the Irrigation and Drainage Division are:

Committee on Publications: Dean F. Peterson, Jr., Utah State University, Logan, Utah.

Committee on Cooperation with Local Section: S. Mark Davidson, 2432 So. Jasmine St., Denver 22, Colo.

Committee on Consolidation and Progressive Betterment of Old Irrigation Systems: A. Alvin Bishop, Utah State University, Logan, Utah.

Committee on Drainage of Irrigated Lands: Charles R. Maierhofer, 666 Marion St., Denver 18, Colo.

Committee on Ground Water: Harvey O. Banks, Dept. of Water Resources, Sacramento, Calif.

Committee on Irrigation and Drainage Practices in Humid Areas: Marion C. Boyer, 1330 W. Michigan St., Room 377, Indianapolis, Indiana.

Task Group on Water Rights Laws in States in Humid Areas: James I. Seay, Jr., 516 Goodwyn Inst. Bldg., Memphis, Tenn.

Committee on Research: Gerald B. Keesee, Bureau of Indian Affairs, Box 912, Gallup, N. M.

Committee on Water Supply and Conservation: Harry F. Blaney, 215 West 7th St., Los Angeles 14, Calif.

Task Group on Methods of Conserving Water: Arthur E. Bruington, 636 W. Hermosa Ave., San Gabriel, Calif.

Task Group on Consumptive Use of Water by Irrigated Crops and Native Vegetation: Harry F. Blaney.

Task Group on Re-Use of Drainage Water and Water REclamation: Lloyd E. Myers, Jr., Box 815C, Route 2, Tempe, Ariz.

Task Group on Water Management: Charles W. Thomas, 19 Cowper Street, Berkeley, Calif.

Task Group on Water Quality: P. H. McGauhey, 6819 Snowden Ave., El Cerritto, Calif.

These men are constantly in need of help from other society members. Why not volunteer your talents to the one heading the committee that is working in your field of interest?

* * *

The following tentative program for Irrigation and Drainage Division sessions at the Washington Convention is published for your information:

General Theme: "Activities Around the World of Civil Engineers and of Engineering and Construction Firms and Agencies of the United States"

Irrigation and Drainage Division

Theme: "World Wide Picture of Irrigation"

Presiding Officer: N. A. Christensen, Director of School of Civil Engineering, Cornell University; Vice Chairman, Program Sessions Committee, Irrigation and Drainage Division, ASCE.

October 20 P.M.

Theme: "Irrigation in Pakistan, India and Ceylon"
Session Chairman: N. A. Christensen
Speakers:

Royce J. Tipton, Tipton & Kalmbach, Inc, Insurance Bldg., Denver, Colorado.
Topic - "Irrigation and Drainage in West Pakistan"

Alfred C. Ingersoll, Professor of Civil Engineering, California Institute of Technology.
Topic - "Aspects of Irrigation in India" - Tentative

Sol Resnick, Hydrologist, University of Arizona.
Topic - "Irrigation and Drainage Development in the Community Development Program, India" - Tentative

Philip P. Dickinson, Technical Assistant to Senator Clair Engle, United States Senate, Washington, D. C.
Topic - "Humid Zone Irrigation --Ceylon"

October 21 P.M.

Theme: "Irrigation in Iraq, South Africa, Australia, and New Zealand"
Session Chairman: William W. Donnan, Drainage Engineer, Agricultural Research Service, Pomona, Calif. Secretary, Irrigation and Drainage Division, ASCE
Speakers:

Martin R. Huberty, Director, University of California Water Resources Center, Los Angeles, California.
Topic - "Irrigation and Drainage in South Africa, Australia, and New Zealand"

M. R. Lewis and wife, Irrigation Engineer, Carrol E. Bradbury and Associates, Los Altos, Calif.
Topic - "The Heritage of Irrigation in Iraq"

Stanley S. Butler, Associate Professor of Civil Engineering, University of Southern California.
Topic - "Irrigation Systems of the Tigris and Euphrates Valleys in Ancient and Modern Times"

October 22 P.M.

Theme: "Irrigation in Europe, Central and South America"
Session Chairman: Royce J. Tipton
Speakers:

L. S. Willardson, Agricultural Engineer, Western Soil and Water Management Research Branch, Soil and Water Conservation Research Branch, Soil and Water Conservation Research Division, U. S. Department of Agricultural, Logan Utah.
Topic - "Irrigation in Republic of Honduras, Dominican Republic and Puerto Rico"

William W. Donnan, Drainage Engineer, Agricultural Research Service, Pomona, California.
Topic - "Drainage and Water Management in Western Europe"

J. E. Christiansen, Professor of Civil Engineering, Utah State University, Special consultant on water development practices for Food & Agriculture Organization of United Nations.
Topic - "Irrigation and Drainage Problems in Uruquay"

* * *

The Irrigation and Drainage Division sponsored three technical sessions at the May convention of ASCE in Cleveland, Ohio. An important part of the program was the report on the Research Committee, headed by G. B. Keesee, on the recent inquiry into research needs in our fields of activity. This committee will organize the material into a symposium type report and probably submit it for publication. This report should be very helpful in pointing up the needs for research and stimulate educational, state, federal and private agencies to the kind of effort needed to guide irrigation and drainage development in the future.

* * *

The Committee on Irrigation and Drainage Practices in Humid Areas also met in Cleveland. Principal items of business were to prepare an outline for a report, "Irrigation and Drainage Practices in Humid Areas," and to plan for sessions to be sponsored by this committee at the New Orleans, Phoenix and Omaha conventions of the society. The following statement on the proposed report is quoted verbatim from the report of Committee Chairman M. C. Boyer:

"For almost as long as man has been an agriculturist he has practiced irrigation in those areas where the natural precipitation is insufficient in quantity or improperly distributed in season to provide his crops the moisture which they require. In modern times he has discovered that in humid areas, where natural moisture is usually sufficient for crop growth, the application of supplemental irrigation water will materially increase crop yield or permit

diversification, to his economic betterment. Conversely, he must at times remove from the land the excess water resulting from storms. The efficient combination of irrigation and drainage of cultivated humid-area lands will bolster crop yields to meet the ever-increasing needs for food and fiber by a rapidly expanding world population.

"Essential to any consideration of the utilization of water for irrigation in a watershed is the necessity of having accurate data on its quantity and quality and the magnitude of the various other uses to which it is put. Further, a basic hydrologic plan for obtaining irrigation water is a must. These are the responsibility of the civil-hydrologic engineer.

"The delivery of irrigation water to the land and the removal of storm excess require many structures, including dams, canals, control works, walls and pumps, floodways and levees. The design of all such structures is within the province of the civil-hydraulic engineer.

"There is a community of interest between the civil and the agricultural engineer. The latter operates directly on the land, designing ditch and tile drainage, contouring the land for conservation of moisture and prevention of erosion and, if irrigation is practiced, designing the system most suitable to the land and crops. It is the civil engineer's responsibility to cooperate with the agricultural engineer, but to deal in the larger aspect, designing structures to gather and remove the excess waters from groups of farms and to provide irrigation water to large tracts.

"No property may be held or used without statutory regulation. This is particularly important with regard to water, which is a migratory resource, subjected by many users to many uses, some of which, such as irrigation, are consumptive. The rights to water use, just as the rights to any other property, must be determined by legislative and court actions. An inventory of water supply, quality, and demand is essential the determination of proper statutory regulation of its use.

"This manual will have served its purpose if it presents criteria and guide lines for the practice of irrigation and drainage in humid areas, including procedures for the collection and evaluation of data on weather, soils, and crops, and water supply, quality, and demand; the development of irrigation water use plans in relation to other uses; the design of structures for the conveyance of irrigation water to the land and the removal of storm excess; and assistance to courts and legislatures in preparing statutory regulations for the protection of water use as a property right."

Tentative Table of Contents of the Report

Chapter 1. Characteristics of humid areas
2. Climate in humid areas
3. Sources of irrigation water
4. Inventory of water resources
5. Soils and crops
6. Maintenance of optimum soil moisture
7. Methods of applying irrigation water
8. Removal of storm water
9. The effects of humid-area irrigation
10. Guides to water-right laws
11. Irrigation and the watershed development program M. C. Boyer of the Indiana Flood Control and Water Resources Commission is chairman of the Committee.

1959-60 officers of the Irrigation and Drainage Division of the Colorado Section are:

Harris R. McDonald, Chairman
Walter C. Ohlsen, Vice-Chairman
Fred A. Houck, Secy.-Treas.

* * *

We have been asked to announce the availability of a new book, "Open-Channel Hydraulics," which is available from McGraw-Hill Book Company, Inc. Author is Prof. Ven Te Chow, M.Asce, of the University of Illinois. It is intended for use by practicing hydraulic engineers as well as for advanced undergraduate and graduate students of hydraulics, irrigation and drainage.

* * *

The Newsletter Editor appreciates the "slightly" increasing number of news items you are sending to him. Your Newsletters would be more interesting to prepare and to read if more of the important news of the day appeared in it. You can help by sending irrigation and drainage news notes to the editor.

* * *

EARLY TRANSACTIONS VOLUMES OBTAINABLE

The feasibility of reproducing the first ten volumes of ASCE Transactions (1872-1881) has been studied. It has been decided that these historic volumes could be reproduced at a cost that would permit a top price of $150 for the ten-volume set. If more than 100 engineers, or libraries, indicate an interest in obtaining such a set, the project will be undertaken. If the endeavor is successful, other rare volumes of Transactions will be reprinted.

Engineers interested in obtaining the ten-volume set should write to the Executive Secretary of ASCE, 33 West 39th Street, New York 18, N. Y.

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